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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Volume 33

April 1964-February 1966

U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

Contents of Volume 33

Page

No. 1, April 1964:

Effect of linseed oil coatings on resistance of concrete to scaling, by William E. Grieb and Roger Appleton . . 1

Evaluation of a new modal split procedure, by Arthur B. Sosslau, Kevin E. Heanue, and Arthur J. Balek 5

Table: State legal maximum dimensions and weights of motor vehicles compared with AASHO standards (text on p. 19) 10

No. 2, June 1964:

The discovery process in highway land acquisition, by Sidney Goldstein, James Rice, and William Lavelle 21

An evaluation of partial taking of property for right-of-way, by George V. Broderick and Floyd I. Thiel 38

No. 3, August 1964:

Selecting the best scale for in-motion weighing, by Russell E. Puckett 45

Effect of moisture on bituminous pavements in Rocky Mountain areas, by William L. Eager 48

Stability of bituminous pavements related to aggregate characteristics, by Robert E. Olsen 54

No. 4, October 1964:

Natural weathering of concrete specimens prepared with cements used in the long-time study, by William E. Grieb and George Werner 57

139 million drivers in 1980, by Edwin M. Cope and Arlene R. Mundy 68

Errata (see vol. 32, No. 9, p. 51) 80

No. 5, December 1964:

Timesharing between two driving tasks—simulated steering and recognition of road signs, by Burton W. Stephens and Richard M. Michaels 81

Evaluation of laboratory vane shear test, by Roger D. Gonghnour and John R. Sallberg 89

Evaluation of results of laboratory vane shear tests with pore-pressure measurements, by Nyal E. Wilson 94

No. 6, February 1965:

Shrink-swell potential of soils, by George W. Ring III . . 97

Friction reducing mediums for rigid pavement subbases, by Albert G. Timms 106

Simplified procedures for determining travel patterns evaluated for a small urban area, by Constantine Ben, Richard J. Bouchard, and Clyde E. Sweet, Jr. 112

Page

No. 7, April 1965:

Headlight glare and median width—three exploratory studies, by Lawrence D. Powers and David Solomon 125

Moving picture technique for highway signing studies—an investigation of its applicability, by Richard D. Desrosiers 143

Motor-vehicle travel estimates—1963 United States; 1962 State-by-State, by Theodore S. Dickerson, Jr., and Ivano E. Corvi 148

No. 8, June 1965:

Highway interchange area development, by Floyd I. Thiel . 153

Aerial color photography and its use in materials surveys, by Jesse R. Chaves and Robert L. Schuster 167

Effectiveness of sign background reflectorization, by Lawrence D. Powers 172

Shear loads on pavements, by Edward S. Barber 179

No. 9, August 1965:

Highway income, expenditures, and highway-user earnings in 46 standard metropolitan statistical areas, by Stanley F. Bielak and James F. McCarthy 185

Table: State legal maximum dimensions and weights of motor vehicles compared with AASHO standards (text on p. 200) 192

No. 10, October 1965:

Abrasion resistance of bridge paints for use in Alaska—field and laboratory tests evaluated, by Bernard Chaiken . 201

Voids, permeability, and film thickness related to asphalt hardening, by Joseph F. Goode and Lawrence A. Lufsey 214

No. 11, December 1965:

Attitudes of drivers determine choice between alternate highways, by Richard M. Michaels 225

Illumination variables in visual tasks of drivers, by H. Richard Blackwell, Richard N. Schwab, and B. S. Pritchard . 237

No. 12, February 1966:

Effect of soil blankets and preloading on settlement, by Edward S. Barber 249

An interpretive review, mechanisms of soil-lime stabilization, by Sidney Diamond and Earl B. Kinter 260

Experimental isolation of drivers' visual input, by Donald A. Gordon 266

Travel habits in cities of 100,000 or more, by Mrs. Thurley A. Bostick and Thomas R. Todd 274

List of Authors

(and volume page references)

	Page		Page		Page		Page
Appleton, Roger	1	Corvi, I. E.	148	Lavelle, William	21	Schuster, R. L.	167
Balek, A. J.	5	Desrosiers, R. D.	143	Lufsey, L. A.	214	Schwab, R. N.	237
Barber, E. S.	179, 249	Diamond, Sidney	260	McCarthy, J. F.	185	Solomon, David	125
Ben, Constantine	112	Dickerson, T. S., Jr.	148	Michaels, R. M.	81, 225	Sosslau, A. B.	5
Bielak, S. E.	185	Eager, W. L.	48	Mundy, A. R.	68	Stephens, B. W.	81
Blackwell, H. R.	237	Goldstein, Sidney	21	Olsen, R. E.	54	Sweet, C. E., Jr.	112
Bostick, (Mrs.) T. A.	274	Goode, J. F.	214	Powers, L. D.	125, 172	Thiel, F. I.	38, 153
Bouchard, R. J.	112	Gordon, D. A.	266	Pritchard, B. S.	237	Timms, A. G.	106
Broderick, G. V.	38	Gonghnour, R. D.	89	Puckett, R. E.	45	Todd, T. R.	274
Chaiken, Bernard	201	Grieb, W. E.	1, 57	Rice, James	21	Werner, George	57
Chaves, J. R.	167	Heanue, K. E.	5	Ring III, G. W.	97	Wilson, N. E.	94
Cope, E. M.	68	Kinter, E. B.	260	Sallberg, J. R.	89		

VOL. 33, NO. 1 APRIL 1964

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WASHINGTON

Interchange of Congress Street Expressway and Northwest Expressway, Chicago, Ill., showing Congress Street Expressway going under the Post Office Building. Halsted Street is in foreground.

IN THIS ISSUE

Effect of Linseed Oil Coatings on Resistance of Concrete to Scaling, *by W. E. Grieb and Roger Appleton* 1

Evaluation of a New Modal Split Procedure, *by A. B. Sosslau, K. E. Heanue, and A. J. Balek* 5

Motor Vehicle Size and Weight Limits (AASHO table on pages 10–11) 19

New Publications 20

TITLE SHEET, VOL. 32

The title sheet for vol. 32, Apr. 1962–Feb. 1964, of PUBLIC ROADS magazine is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by a request to the editor of the magazine, Bureau of Public Roads, Washington, D.C., 20402.

U.S. DEPARTMENT OF COMMERCE
LUTHER H. HODGES, Secretary

BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

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Effect of Linseed Oil Coatings on Resistance of Concrete to Scaling

BY THE MATERIALS RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported by[1] WILLIAM E. GRIEB, Highway Research Engineer, and ROGER APPLETON, Supervisory Engineering Technician

This article reports the results of a study of the use of linseed oil surface coatings to prevent scaling caused by the use of de-icing chemicals. These tests were made because of the wide variation in the State specifications governing the use of this material. It is a continuation of tests by the Bureau of materials and procedures for protecting concrete pavements against scaling caused by the use of deicing chemicals. These tests are being continued. In this investigation four different linseed oil surface coatings were used on air-entrained and non-air-entrained concrete with high and low slumps.

The data presented show that for non-air-entrained concrete, all of the linseed oil coatings were of some benefit in protecting the concrete surfaces from scaling. However for the air-entrained concrete, only two of the surface coatings tested were beneficial.

Introduction

MANY State highway departments have been using surface coatings to protect concrete pavements from scaling and disintegration caused by the use of de-icing chemicals. These coatings are often applied to concrete placed late in the fall to give it added protection and to compensate for the short aging period before the concrete is subjected to freezing. One of the materials often used for surface protective coatings is linseed oil. The specifications governing the use of this material vary. Some States specify its use as an emulsion and others require that it be diluted with mineral spirits when applied. One or two applications of the linseed oil may be required.

Tests were made in the laboratory of the Bureau of Public Roads to determine whether the use of the surfaee treatments of linseed oil are beneficial in preventing scaling of the concrete. In these tests, four different concrete mixes were used. They were non-air-entrained low slump concrete and non-air-entrained high slump concrete; air-entrained low slump concrete, and air-entrained high slump concrete. Five different surface treatments were used on concrete slabs. The control slabs were given no surface treatment, and the test slabs were treated respectively with two coats of boiled linseed oil, two coats of raw linseed oil, one coat of boiled linseed oil, or two coats of linseed oil emulsion.

Conclusions

The results of the tests made in the laboratory of the Bureau of Public Roads warrant the following conclusions:

- For non-air-entrained concrete, each of the linseed oil surface treatments tested was of some benefit in preventing or delaying surface scaling on the test slabs.
- For air-entrained concrete, only two of the four linseed oil surfaee treatments were beneficial. These beneficial treatments were two applications of either boiled or raw linseed oil. One application of the boiled linseed oil or the two applications of the linseed oil emulsion were of no benefit in preventing or delaying scaling on the test slabs.
- When two applications of the linseed oil were used, the concrete had slightly better overall resistanee to scaling when boiled linseed oil was used.
- As would be expected, these tests with linseed oil also showed that slump affected the resistance of conerete to scaling. Both the air-entrained and non-air-entrained concrete having a low slump had equal or better resistanee to scaling than similarly treated concrete having a high slump.
- The air-entrained concrete had equal or better resistance to scaling than the similarly treated non-air-entrained concrete.

The pattern of scaling developed on the different groups of slabs during the freezing and de-icing tests is shown in figure 5. These conclusions are most readily apparent when related to this figure.

Mix Data

Four different mixes were used for the concrete test slabs. The mixes were: Non-air-entrained concrete having a 2½-inch slump, non-air-entrained concrete having a 6-inch slump, air-entrained concrete having a 2½-inch slump, and air-entrained concrete having a 6-inch slump. The mix data for the four mixes are given in table 1. All mixes contained approximately 6 bags of cement per cubic yard of concrete. The air content for the air-entrained concrete was 5 percent. For both the air-entrained and non-air-entrained mixes, the 6-inch slump concrete contained 0.7 gallon of water per bag of cement more than the corresponding 2½-inch slump concrete.

Materials

The same materials were used for all mixes. They were a type I portland cement having an equivalent alkali content of 0.6 percent, a siliceous sand having a fineness modulus of 2.75, and a crushed limestone uniformly graded from 1 inch to No. 4. When needed, a commercially available aqueous solution of neutralized Vinsol resin was used to entrain air. The linseed oils used met Federal Specifications.

Curing

The test specimens were concrete slabs 16 by 24 inches by 4 inches in depth and having a raised edge or dam around the perimeter of the top surface. The specimens were similar to those described in a previous article in PUBLIC ROADS on resistance of concrete surfaces to scaling.[2] The slabs were cast in watertight molds that had metal bases. The top surfaces of the slabs were screeded with a wooden straightedge then, about 3 hours after the molding, the slab surfaces were given a light broomed finish. This finish was similar to that given a concrete pavement. All specimens were molded, finished, and cured, in the same manner. All slabs were cured in the molds with wet burlap for 2 days, ponded for 12 days, and then dried in laboratory air at 70° to 80° F. and 30 percent relative humidity for 7 days prior to the application of the surfaee treatments. After the surface treatments had been applied, the slabs were kept in laboratory air for 7 to 14 days and then placed in the outdoor exposure area. When two applications of a surface treatment were used, they were applied 24 hours apart. All specimens were from 28 to 35 days old

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

[2] *Resistance of Concrete Surfaces to Scaling by De-Icing Agents*, by William E. Grieb, George Werner, and Donald O. Woolf, PUBLIC ROADS, vol. 32, No. 3, August 1962, pp. 64-73.

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Table 1.—Mix data [1]

Proportions by dry weight	Cement content	Water content	Slump	Air content	Weight of plastic concrete
Pounds	*Bags/cu. yd.*	*Gal./bag*	*Inches*	*Percent*	*Lb./cu. ft.*
94–205–305	6.2	5.5	2.6	2.0	149.1
94–205–305	6.1	6.2	6.0	1.4	148.7
94–190–305	6.2	5.2	2.7	5.0	145.9
94–190–305	6.1	5.9	6.2	5.1	144.2

[1] Materials used were: (1) type I portland cement; (2) crushed limestone, 1-inch maximum size; and (3) siliceous sand, fines modulus of 2.75.

when they were stored in the exposure area and the first natural freeze occurred 3 days later.

Surface Treatments

Ten identical slabs were made from each of the four mixes. Two slabs made for each mix were given one of the following surface treatments: (1) Control slabs, no surface treatment; (2) two applications of boiled linseed oil; (3) two applications of raw linseed oil; (4) one application of boiled linseed oil; and (5) two applications of linseed oil emulsion. The first coating for both the boiled and raw linseed oil treatment was applied as a mixture of equal parts by volume of linseed oil and mineral spirits at a rate of 1 gallon per 40 square yards of surface. The second coat was applied as undiluted linseed oil at a rate of 1 gallon per 67 square yards.

The linseed oil emulsion used was the same as that described in the special provisions of one of the State specifications for surface sealing of bridge decks. It consisted of 1 part boiled linseed oil, 1 part kerosene, 3 parts water plus a small amount of trisodium phosphate, and a small amount of a nondetergent soap powder as an emulsifying agent. The emulsion was applied at a rate of 1 gallon per 10 square yards of surface for each application. Similar emulsions have been used by other State highway departments.

Observations were made to determine the time necessary for the different coatings to be absorbed by the concrete. The boiled and raw linseed oils diluted with mineral spirits were absorbed in about 30 minutes, whereas the linseed oil emulsion was absorbed in about 1 hour and 40 minutes. The rate of absorption varied according to the mix used in the slabs on which the coating was applied. Slabs made with the air-entrained concrete having a high slump absorbed the coating more rapidly than slabs made with the other mixes. Those made with the non-air-entrained concrete having high slump absorbed the coating the next fastest, followed by those made with the air-entrained low-slump concrete. Slabs made with the non-air-entrained low-slump concrete took the longest time to absorb the coatings. A longer period of time was required for absorption of the second coating by all of the slabs. This information may be of value as a guide for the use of these coating materials in the field.

Testing Procedure

Each evening when freezing was expected, the top surface of each slab was covered with one-fourth to one-half inch of water. The next morning after the water had frozen, flake calcium chloride was spread uniformly over the ice-encrusted surface at a rate of about 2.4 pounds per square yard of surface. Although this rate of application was greater than would be used in the field, this is the rate of application that has been used on other research projects. After the ice had melted, the surface was washed and fresh water was left on the surface. The specimens were examined periodically and were rated by visual observations according to the amount and depth of the scaling. A general description of the numerical ratings is as follows:

0–no scale.

1–scattered spots of very light scale.

2–scattered spots of light scale with mortar surface above coarse aggregate removed.

Table 2.—Rating of slabs

Surface coatings	Rating [1] after freezing and thawing for cycles—						
	10	20	30	45	60	80	105
Non-air-entrained concrete—2.6-in. slump, 2.0 percent air:							
None	4	9	10				
2 coats boiled linseed oil	0	0	0	0	0	0	1
2 coats raw linseed oil	0	0	0	0	1	1	1
1 coat boiled linseed oil	0	0	0	1	2	2	4
2 coats linseed oil emulsion	0	0	0	2	4	5	6
Non-air-entrained concrete—6.0-in. slump, 1.4 percent air:							
None	8	10					
2 coats boiled linseed oil	0	0	1	1	1	1	2
2 coats raw linseed oil	0	0	1	1	3	4	4
1 coat boiled linseed oil	1	4	9	10			
2 coats linseed oil emulsion	0	2	5	7	9	9	9
Air-entrained concrete—2.7-in. slump, 5.0 percent air:							
None	0	0	0	0	1	1	3
2 coats boiled linseed oil	0	0	0	0	0	0	1
2 coats raw linseed oil	0	0	0	0	0	0	1
1 coat boiled linseed oil	0	0	0	1	2	3	4
2 coats linseed oil emulsion	0	0	0	0	1	2	3
Air-entrained concrete—6.2-in. slump, 5.1 percent air:							
None	0	0	0	0	3	4	6
2 coats boiled linseed oil	0	0	0	0	0	0	2
2 coats raw linseed oil	0	0	0	1	3	3	4
1 coat boiled linseed oil	0	2	4	5	6	6	5
2 coats linseed oil emulsion	0	1	1	1	4	4	5

[1] Each rating is an average for two test specimens. Rating of 0 indicates no scaling; rating of 5 indicates a significant amount of scaling; rating of 10 indicates deep scaling over entire surface of specimens.

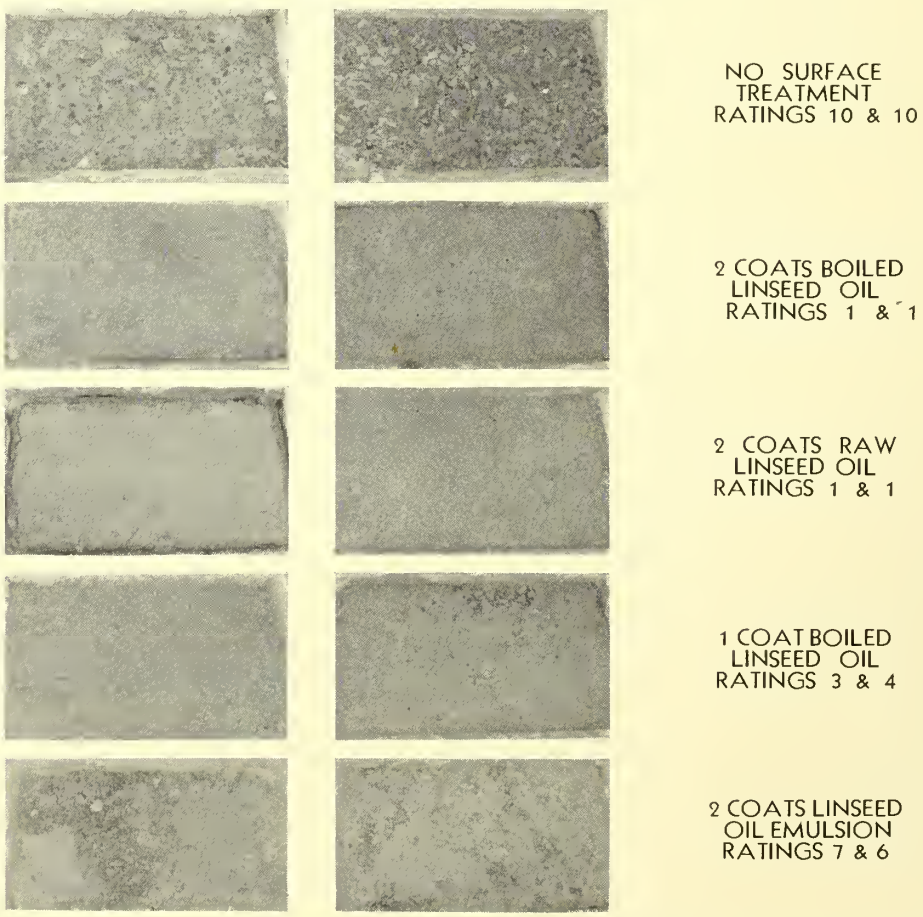
NO SURFACE TREATMENT RATINGS 10 & 10
2 COATS BOILED LINSEED OIL RATINGS 1 & 1
2 COATS RAW LINSEED OIL RATINGS 1 & 1
1 COAT BOILED LINSEED OIL RATINGS 3 & 4
2 COATS LINSEED OIL EMULSION RATINGS 7 & 6

Figure 1.—Effect of linseed oil surface treatments on scaling—105 cycles, 2.6-in. slump, 2.0 percent air.

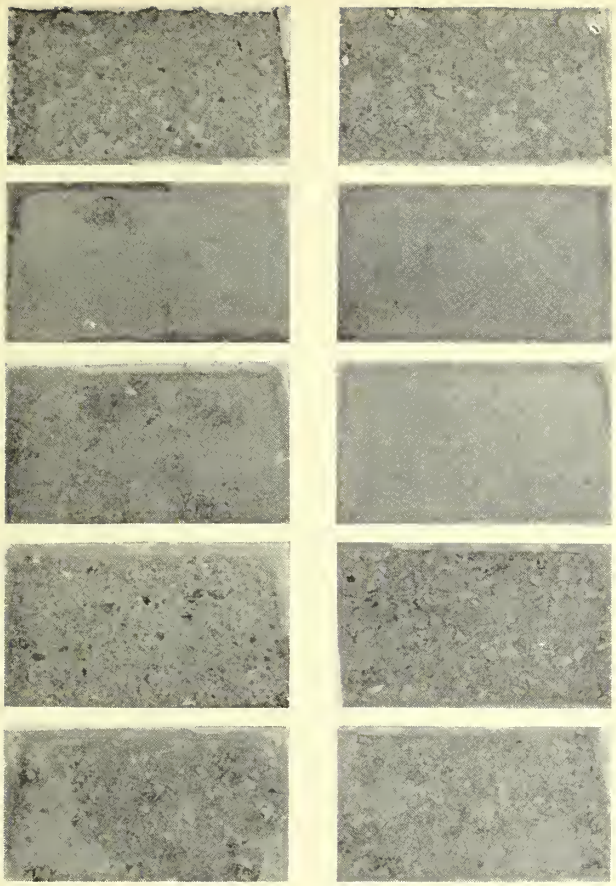
NO SURFACE TREATMENT RATINGS 10 & 10
2 COATS BOILED LINSEED OIL RATINGS 3 & 1
2 COATS RAW LINSEED OIL RATINGS 6 & 1
1 COAT BOILED LINSEED OIL RATINGS 10 & 10
2 COATS LINSEED OIL EMULSION RATINGS 9 & 9

Figure 2.—Effect of linseed oil surface treatments on scaling—105 cycles, 6.0-in. slump, 1.4 percent air.

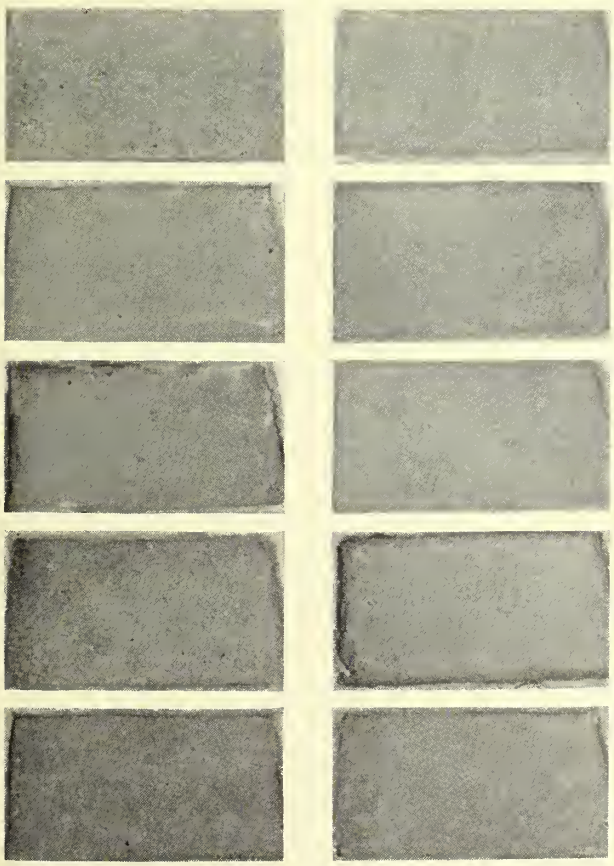
NO SURFACE TREATMENT RATINGS 3 & 3
2 COATS BOILED LINSEED OIL RATINGS 1 & 1
2 COATS RAW LINSEED OIL RATINGS 1 & 1
1 COAT BOILED LINSEED OIL RATINGS 4 & 3
2 COATS LINSEED OIL EMULSION RATINGS 3 & 3

Figure 3.—Effect of linseed oil surface treatments on scaling—105 cycles, 2.7-in. slump, 5.0 percent air.

3—light scale over about one-half of the surface.

4—light scale over most of the surface.

5—light scale over most of the surface, with a few moderately deep spots, where the mortar surface was below the upper surface of the coarse aggregate.

6—scattered spots of moderately deep scale.

7—moderately deep scale over one-half of the surface.

8—moderately deep scaling over entire surface.

9—scattered spots of deep scale with the mortar surface well below the upper surface of the coarse aggregate; otherwise moderately deep scaling.

10—deep scale over entire surface.

A rating of 5 or more would indicate significant or major scaling. The ratings given the slabs were based on the judgment of different observers at the various times that the observations were made, which accounts for occasional slight reversals. The rating scale used was the same as that given in the previous article (see footnote 2).

These specimens were in the outdoor exposure area for two winters and had a total of 105 cycles of freezing and thawing (45 cycles the first winter and 60 cycles the second). Previous tests have shown that exposure for two winters is sufficient to indicate the resistance of concrete test specimens to scaling caused by the use of calcium chloride.

Test Results

Non-air-entrained-concrete, low slump

The average ratings of the slabs after 10, 20, 30, 45, 60, 80, and 105 cycles of freezing and thawing are given in table 2. These data show that for the non-air-entrained concrete having a low (2.6-inch) slump, when the slabs had no surface coatings, their ratings were 10 after 30 cycles of freezing and thawing. The tests on these slabs were then discontinued. None of these slabs coated with the linseed oil showed any sign of scaling at 30 cycles of freezing and thawing. At 45 cycles, scaling had started on the slabs that had received one coat of boiled linseed oil and on those treated with the two coats of the emulsion. At 105 cycles, the slabs treated with two coats of the emulsion showed severe scaling over part of the surface and were given an average rating of 6. The slabs treated with one coat of boiled linseed oil showed light scaling over the entire surface and were given a rating of 4. The slabs that had received two coatings of boiled or raw linseed oil showed very little scaling and were given a rating of 1.

Non-air-entrained concrete, high slump

For the non-air-entrained concrete having a high slump (6 inches), the slabs that had received no surface treatment were completely scaled after only 20 cycles of freezing and thawing and were given a rating of 10. For this same number of cycles, the slabs treated with one coat of boiled linseed oil and those treated with two coats of the emulsion had ratings of 4 and 2, respectively. Those re-

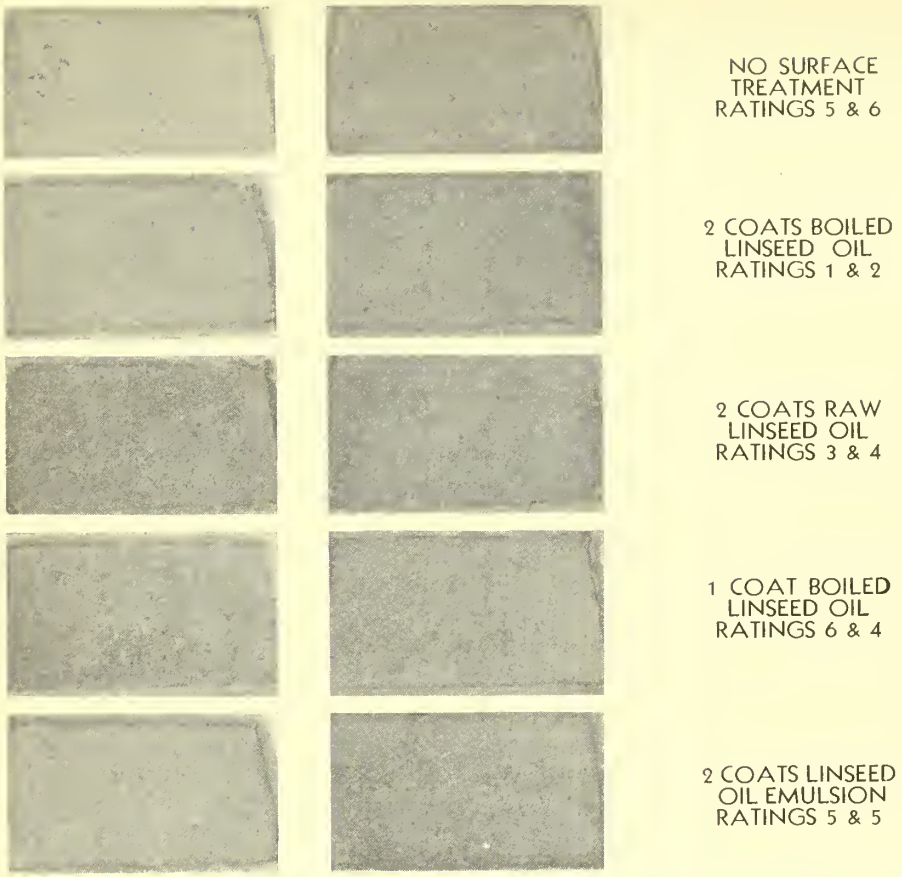
NO SURFACE TREATMENT RATINGS 5 & 6
2 COATS BOILED LINSEED OIL RATINGS 1 & 2
2 COATS RAW LINSEED OIL RATINGS 3 & 4
1 COAT BOILED LINSEED OIL RATINGS 6 & 4
2 COATS LINSEED OIL EMULSION RATINGS 5 & 5

Figure 4.—Effect of linseed oil surface treatments on scaling—105 cycles, 6.2-in. slump, 5.1 percent air.

ceiving two coatings of raw and boiled linseed oil showed no signs of scaling at that time. After 45 cycles, deep scaling over the entire surface (rating of 10) was found on the slabs treated with one coat of boiled linseed oil. Those covered with two coats of the emulsion had a rating of 9 after 60 cycles. After 105 cycles, the slabs treated with two coats of the boiled linseed oil had an average rating of 2, and those treated with two coats of raw linseed oil had an average rating of 4. There was a marked difference in the amount of scaling between the two slabs given the two coats of the raw linseed oil. One slab had virtually no scaling and was given a rating of 1. The other slab was severely scaled over a portion of its surface and was rated at 6. No reason can be given for the difference in performance of these presumably identical slabs.

Air-entrained concrete, low slump

All of the slabs prepared with air-entrained concrete having a low slump showed good resistance to scaling. After 105 cycles of freezing and thawing, the slabs covered with two applications of the boiled or raw linseed oils showed very little scaling and were rated as 1. The average rating for all the other slabs was 3 or 4, these ratings applied to both the treated and the untreated reference (control) slabs.

Air-entrained concrete, high slump

The slabs prepared with air-entrained concrete having a high slump showed good to fair resistance to scaling. The slabs that had two coatings of the boiled linseed oil had the best resistance, having a rating of 2 after 105 cycles of freezing and thawing. Slabs treated with two coatings of the raw oil had a rating of 4 after 105 cycles. There was little difference in the surface condition of the other slabs, they all had average ratings of 5 or 6.

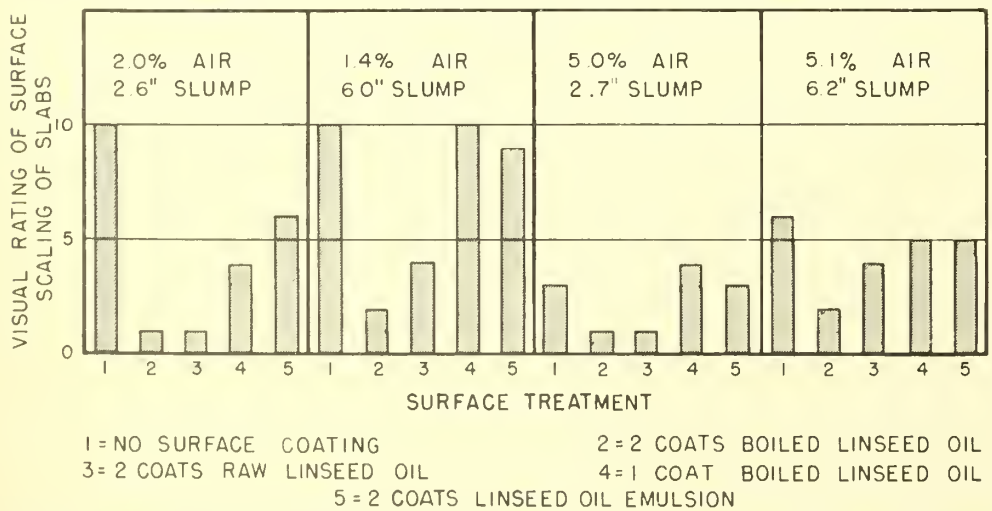
2.0% AIR
2.6" SLUMP
1.4% AIR
6.0" SLUMP
5.0% AIR
2.7" SLUMP
5.1% AIR
6.2" SLUMP
VISUAL RATING OF SURFACE SCALING OF SLABS
0
5
10
1 2 3 4 5 1 2 3 4 5 1 2 3 4 5 1 2 3 4 5
SURFACE TREATMENT
1 = NO SURFACE COATING
2 = 2 COATS BOILED LINSEED OIL
3 = 2 COATS RAW LINSEED OIL
4 = 1 COAT BOILED LINSEED OIL
5 = 2 COATS LINSEED OIL EMULSION

Figure 5.—Effect of linseed oil surface coatings on scaling after 105 cycles of freezing and thawing with calcium chloride.

Condition of all specimens

Photographs of all the slabs after 105 cycles of freezing and thawing, or when they were given a rating of 10, are shown in figures 1 to 4, inclusive. These figures also show the final rating of each slab. There was, in general, good uniformity between the two slabs from the same mix given the same surface treatment. Only for one pair of slabs made with non-air-entrained concrete having a high slump, as previously mentioned, was the difference in ratings between the two similar slabs greater than two. For more than half of the similar pairs of slabs, the ratings given were the same.

Summary

A summary of the ratings of the slabs after 105 cycles of freezing and thawing is shown in figure 5. This figure shows that, for both the low and the high slump non-air-entrained concretes, all of the linseed oil surface treatments were beneficial in preventing or delaying scaling caused by the use of de-icing chemicals. Applications of two coats of either the boiled or the raw linseed oil were the most beneficial surface treatments. With one exception, the slabs given the two coats of the boiled or raw linseed oil were the only ones of the non-air-entrained concretes that did not show significant scaling after 105 cycles of freezing and thawing.

For the air-entrained concrete having either low or high slump, the two-coat application of either the boiled or the raw linseed oil was the only surface treatment effective in preventing or delaying scaling. The other surface treatments were of little or no benefit. No significant scaling occurred on any slabs prepared with the low-slump, air-entrained concrete, including those that received no surface treatment. All of the slabs except those given the two coats of the boiled or raw linseed oil showed significant scaling when they had been prepared with the high-slump air-entrained concrete.

The slabs treated with two coats of the boiled linseed oil were equally or more resistant to scaling than those treated with two coats of the raw linseed oil.

Greater resistance to scaling was furnished by the low-slump concrete than by the corresponding high-slump concrete, and greater resistance was furnished by the air-entrained concrete than by the corresponding non-air-entrained concrete.

Evaluation of a New Modal Split Procedure

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

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A Bureau of Public Roads evaluation of a new modal split technique of interest to urban transportation planners responsible for estimating future public transit requirements is presented in this article. The new modal split technique was developed by the Traffic Research Corporation for the use of the National Capital Transportation Agency in estimating 1980 transit requirements for the Washington, D.C., area. Because sound estimates of transit patronage are required for the development of comprehensive urban transportation plans, Pubic Roads conducted a two-phase test of this procedure.

In the first phase, the effectiveness of the new modal split procedure to reproduce a known situation was tested. In the second phase, the sensitivity of the procedure was assessed; that is, its ability to reflect changes in input variables.

The evaluation tended to confirm the usefulness of this new modal split procedure but also revealed limitations that should be considered before further application. Comments of three transportation planning officials on the evaluation and the findings therefrom have also been included with this article.

Introduction

AN EVALUATION of a new technique developed for estimating the relative use of the private and public modes of transportation is presented in this article. The development of comprehensive urban transportation plans requires sound estimates of transit patronage. The Traffic Research Corporation (TRC) under contract to the National Capital Transportation Agency (NCTA) developed a new modal split procedure for estimating the relative usage of the private and public modes of transportation. This procedure was utilized by the NCTA in developing a 1980 transportation plan for the Washington, D.C. area, prior to this Public Roads evaluation. This evaluation of the modal split procedure was made to gain insight into its accuracy and to provide potential users with a quantitative analysis of a research application.

The evaluation project conducted by the Bureau of Public Roads consisted of two distinct phases: (1) A test of the modal split technique as a means of reproducing a known situation; specifically the transit usage reported in the 1955 Washington, D.C., origin and destination survey; and (2) a test of the sensitivity of this new modal split procedure to changes in the input parameters.

Background

The relationship of the usage of private automobiles and of public transportation is growing in importance, particularly in large cities. Although useful planning techniques were available for use in developing comprehensive urban transportation plans, the ultimate in such planning techniques has not been attained. Of the techniques that have been developed one relates the proportion of use of public transportation to car ownership and population density; and another relates the proportion of use of public transportation to some function of the traveltime required for transit and automobile travel.

Many factors influence the choice of a mode of transportation, according to different studies reported. One study conducted in Cook County, Ill., in 1957, revealed that 32.4 percent of travelers consider time the most important factor in choosing a mode of travel. Other prime factors reported were comfort, 17.4 percent; cost, 5.3 percent; and walking distance, 8.0 percent. A car was reported to be a requisite for transportation by 12.5 percent of those surveyed, and 12.8 percent reported they had no choice other than public transportation. Miscellaneous factors were reported by 11.6 percent (*1*).[3]

Modal Split Technique

Two basic approaches to the modal split technique are possible. In one, the split between private and public transportation trips for each zone is estimated and the transit and automobile trips are distributed separately between zones. In the other approach—the one used by the new modal split procedure—the split is considered after the distribution of total person movements between zones has been made.

The new modal split technique is basically a diversion curve procedure in which relative transit usage is related to five selected variables. These variables are: (1) The ratio of door-to-door traveltime by public transit to the door-to-door traveltime by private automobile. (2) The ratio of excess traveltime by public transit to excess traveltime by private automobile. This ratio is used as a measure of relative travel service and is referred to as the "service ratio." (3) The ratio of out-of-pocket travel cost by public transit to the out-of-pocket travel cost by private automobile. (4) The economic status of the person making the trip. (5) Trip purpose.

The items considered in the development of the five variables are shown in the following expressions.

$$\text{Traveltime ratio} = \frac{a+b+c+d+e}{f+g+h} \qquad (1)$$

Where,

a = time on transit vehicle
b = transferring time between transit vehicles
c = time spent in waiting for transit vehicle
d = walking time to transit vehicle
e = walking time from transit vehicle
f = automobile driving time
g = parking delay time at destination
h = walking time from parking place to final destination
i = gasoline cost $\left[\frac{\text{gallons}}{\text{mile}} \times \text{distance} \times \frac{\text{cost}}{\text{gallon}}\right]$
j = oil change and lubrication cost (cost of oil change per mile times distance)
k = parking cost at destination
L = number of persons per vehicle

$$\text{Service ratio} = \frac{b+c+d+e}{g+h} \qquad (2)$$

$$\text{Cost ratio} = \frac{\text{transit fare}}{\left(\frac{i+j+\frac{k}{2}}{L}\right)} \qquad (3)$$

Economic status = median income per worker. (4)

Trip purpose = either, home-based work trips or all nonwork trips except those made to school. (5)

To develop the modal split relationships for each trip purpose, determinations were made of the percentage of travelers that

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964, as a report titled, *Test of the Modal Split Procedure Developed by the National Capital Transportation Agency.*

[2] Mr. Sosslau is now employed by the Tri-State Transportation Committee.

[3] References indicated by italic numbers in parentheses are listed on page 17.

used public transit and private automobiles from each origin to each destination. This usage was then related to the four basic determinant factors—traveltime ratio, travel cost ratio, service ratio, and economic status. The trip information, from which these relationships were developed, was obtained from travel surveys made in the Washington area and from supporting evidence gathered in other cities (*2*). All observations from each study were stratified by trip purpose. Next, divisions were made of: The cost ratio into four ranges, the excess time ratio into four ranges, and the income level into five ranges. By multiplying the number of ranges (4×4×5), 80 individual combinations of these three determinant factor ranges were obtained, thereby providing 80 time-ratio diversion curves for each trip purpose. For each of the 80 combinations the observations concerning modal split were plotted against the traveltime ratio. Figure 1 shows four of these curves.

Information for curve development was obtained from the 1955 Washington, D.C., origin and destination survey and from the 1961 Federal employee survey. For the work-trip relationships, those trips arriving at zero sector destinations (see figure 2) between 6:54 a.m. and 9:06 a.m. were analyzed. For nonwork, nonschool trips the data studied were for the period 9:12 a.m to 3:45 p.m. Selected trips to nonzero sector destinations were also used to supplement the zero sector oriented data. Data for 1955 and 1961 were combined, after adjustments had been made to put the two sets of information on an equal basis, and average grouped points calculated to obtain one set of relationships. As sufficient information upon which to base the relationships was not available from Washington, D.C., travel survey data from Toronto, Canada, and Philadelphia, Pennsylvania, were used to supplement it (*2*). Basically, these data were necessary to extrapolate the curves developed from Washington data for the 1980 estimate because: (1) Little information was available for Washington that showed traveltime ratios of less than one, and (2) little information was available that showed cost ratios of less than 0.5. A computer program was developed to apply the modal split procedure. Briefly, this program has been written for a 7090 computer in the FORTRAN language. A complete description of the modal split relationship development can be obtained from references *2* and *6*.

Summary of Findings

The Public Roads test of the new modal split technique developed by TRC for use of the NCTA to estimate 1980 transit usage in Washington, D.C., shows that it may be a useful tool for forecasting transportation system usage. Although the tests confirmed its usefulness, they also revealed limitations that should be considered before further application of this particular modal split technique. A more accurate and useful tool may be developed upon further investigation and analysis of this method.

The results of the BPR test against 1955 O–D data indicate that the technique can reasonably reproduce the conditions from which the modal split relationships were developed. The estimate of transit work trips to the zero sector from the entire area obtained by use of the procedure was as good as could be expected. This estimate of transit trips was within one root-mean-square error of the 1955 O–D survey estimate. Total nonzero sector destined transit work trips were less accurately estimated by the modal split procedure, probably because the curves were developed almost entirely from zero sector oriented trips. Additional research many well indicate that separate sets of curves are required for CBD and non-CBD oriented trips.

The restraint added to the transfer matrix in the test, which eliminated certain non-CBD to non-CBD trips, may be unnecessary if a separate set of non-CBD curves is developed, or if a new set of relationships that indicate zero transit ridership at a traveltime ratio of 5.00, instead of 10.00, is used. The corridor analysis indicated a geographical bias in the modal split estimates for work trips.

The estimate of nonwork transit trips to zero sector destinations from the entire area was also within acceptable limits of accuracy, although it was not as accurate as for work trips. Perhaps this difference in accuracy was caused by the nonwork curves being developed from offpeak time period data and applied to the peak period. Further research is needed to evaluate the application to another time of a set of relationships developed for one time period.

On a corridor basis, estimates for all trips having origins outside the zero sector and destinations in the zero sector were slightly less accurate than the O–D estimate. The analysis of district-to-district trips for both work and nonwork showed the variation between the estimate of transit trips and O–D trips to be less than the expected variation in the O–D trips.

All the variables considered in the test of the modal split technique appear to relate to modal choice. However, this alone does not necessarily indicate a generally applicable procedure. Estimates of necessary input variables must be sufficiently accurate so as to not seriously impair the accuracy of the estimated transit usage. The sensitivity tests showed that substantial weight is given to certain of the input variables that are difficult to estimate. The observed change in the modal split when automobile excess time was varied indicates the high weight placed on this parameter. The 2 minutes added to automobile parking delay and walking times in the CBD had a greater effect on the modal split of trips destined to the CBD than doubling fares, doubling parking costs, factoring transit or highway times by 0.75, or factoring transit times by 1.5. These excess time values are among the most difficult to estimate. The 2-minute increase in excess time (test *C*) is not considered unrealistic in that the mean 1955 CBD excess automobile time used in the development of the curves was 3.6 minutes and the mean excess automobile time estimated for 1980 was 6.7 minutes.

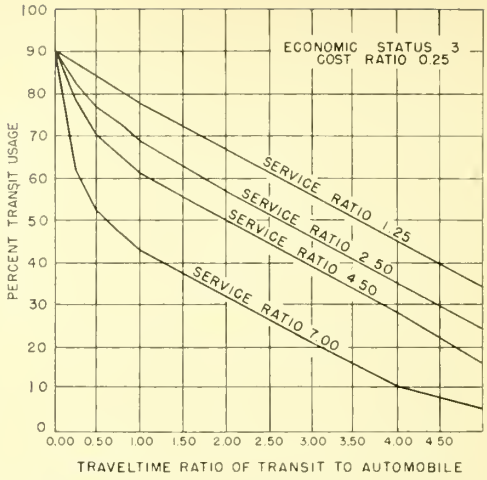

Figure 1.—Example of modal split relationship.

The sensitivity analysis indicated that parameters reflecting 1980 automobile terminal conditions; that is, parking delay, walking time, and parking costs, had a far greater weight in the modal split determination than any of the parameters that reflected the proposed transit system. The range of meaningful cost ratio values was too narrow to permit evaluation of alternate fare structures.

Additional work indicated as being desirable includes: (1) Extension of the cost ratio ranges and level of service ratio ranges to reflect wider variations in system conditions. (2) Better estimating procedures to improve the accuracy of those model inputs that show the greatest sensitivity to modal split. (3) The testing of time differences rather than or in conjunction with time ratios, which may produce greater sensitivity of the procedure to highway and transit system changes.

Test of Modal Split Technique Against 1955 O–D Survey

The modal split technique was used by NCTA for estimating 1980 transit usage on a proposed system. Public Roads developed parameters reflecting highway system and transit system 1955 operating characteristics and applied the modal split procedure to 1955 conditions. The estimated transit usage was then compared to the transit usage reported in the 1955 Washington, D.C., O–D survey. The 160 modal split diversion curves (80 for each purpose) used in the Public Roads test were the same curves used by NCTA in developing the 1980 transit usage estimates.

Two principles were established for the test procedure: (1) The data from the 1955 Washington, D.C., origin and destination survey were to be adhered to as closely as possible in preparing the input parameters. (2) The same procedures used by NCTA in preparing input parameters for the application of the modal split procedure to estimate the 1980 transit usage were to be used.

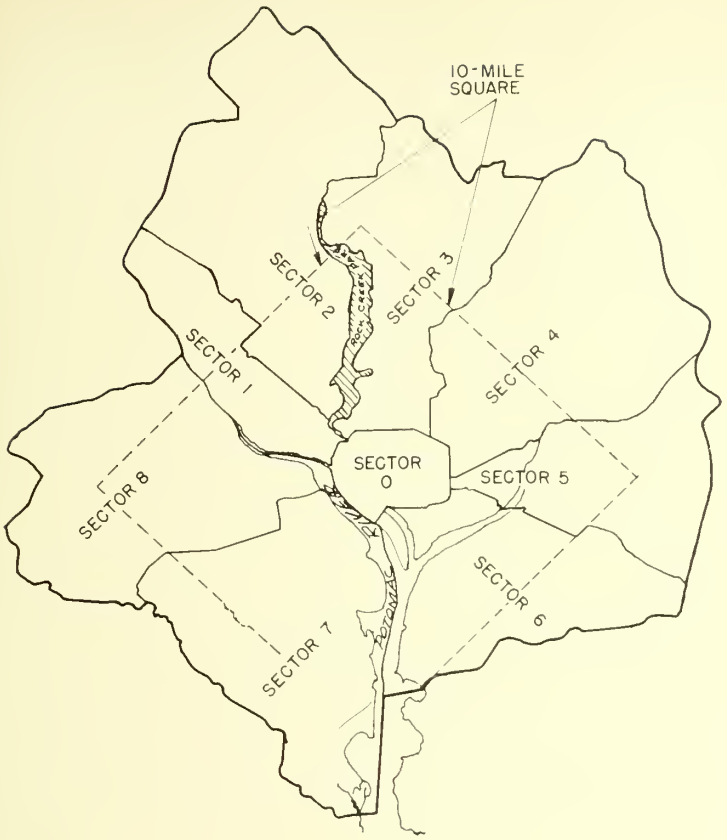

Figure 2.—1955 Washington, D.C. survey sectors.

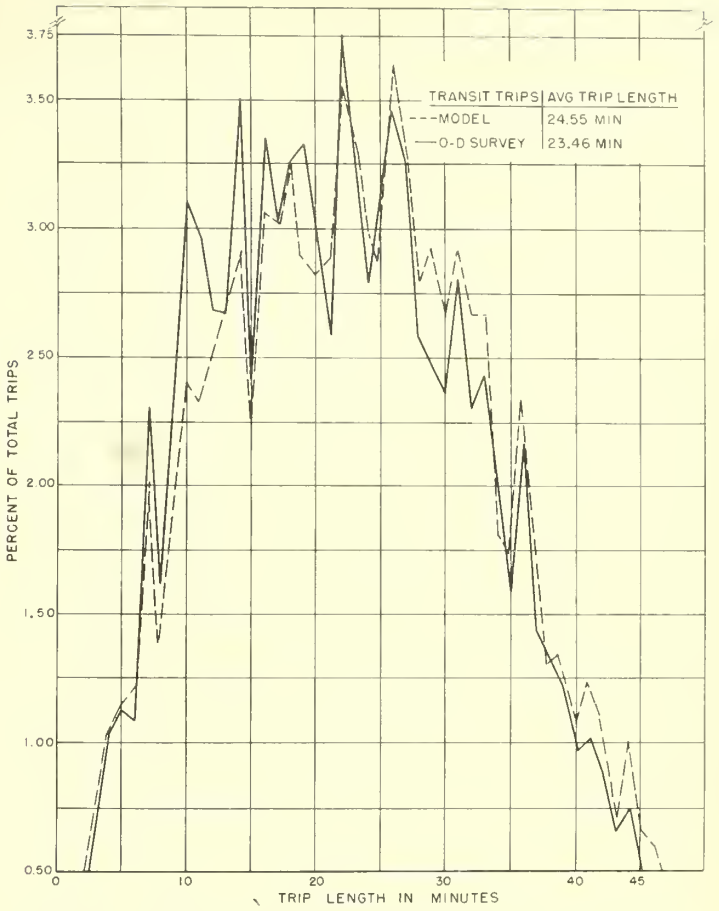

Figure 3.—Trip length frequency distribution comparison.

The modal split technique was tested to determine its ability to predict: (1) Areawide modal split; (2) areawide to Central Business District (CBD) destination (zero sector) modal split; (3) areawide to non-CBD destination modal split; (4) modal split from each of eight survey sectors (corridors) to the CBD; (5) modal split between each of the survey sectors; and (6) modal split between each district in the survey area.

Preparation of Input Data

Generally, data were developed on a zone basis for each of the 400 survey zones considered. Adjustments were made in the zonal data so that the summary of this zonal data by district would closely match the district data used for curve development. This was in accordance with a goal of the test; that is, to evaluate the modal split technique rather than the manner in which the input parameters were prepared.

System Parameters

O-D interchanges

TRC in its development of the modal split relationships reproduced the 1955 O–D trip data in a linked form (*3*). The only deviations from the normal linking process made by TRC are described in the next paragraph.

Linked trips that originated at home but having intermediate change mode or serve passenger purposes and finally destined for home were omitted from the linked trip file. The procedure usually followed is to produce two trips; for example, one from home to personal business and another from personal business to home. All unlinkable, serve passenger, or change travel mode trips were omitted in this test. Normally, each trip is evaluated and a decision reached as to the purpose to be considered. For example, a change mode trip having a destination at an airport—not a linkable trip as the person presumably leaves the area—is usually classified as a business trip.

Only trips made during the peak traffic period of 6:54 a.m. to 9:06 a.m. were considered for this test. However, the modal split nonwork curves were developed from offpeak traffic data for the period of 9:12 a.m. to 3:54 p.m. Work trip curves were developed from data on peak traffic period trips—6:54 to 9:06 a.m. As the modal split model had been used to estimate 1980 transit usage in the a.m. peak traffic period, for this test both sets of curves—work and nonwork—were applied to the peak traffic period.

Two sets of origin and destination survey trip interchanges were developed for the test: trips to and from work, and all nonwork trips except those to and from school. For each set two trip files were required: (1) total person trips—automobile driver; transit passenger; and truck, taxi, and automobile passenger—for input to the modal split program; and (2) trips by transit for comparison with the output of the modal split program.

Highway times and distances

A necessary input to the modal split procedure is the traveltime between zones via the highway network. These times were obtained from the 1955 O–D survey for use in the development of the modal split relationships. For testing the 1980 application of the modal split procedure, times were obtained from "trees" or minimum time paths (*4*). Again, the procedure used by NCTA for their 1980 estimate of transit usage was followed as the guide for this test. A highway network was coded in order to obtain these minimum time paths.

A 1955 highway network was available for use in this testing. However, the times coded on each section of highway derived from 1955 speed runs were more representative of average daily traveltimes than the peak traffic times required. It was necessary to adjust the average daily traveltimes obtained from trees built with this system to match those peak traffic times used for developing the modal split relationships (district to district reportings from O-D survey). The adjusted highway times used in combination with highway distances to obtain a gasoline cost per mile

were determined from minimum time routes obtained in this process.

Transit time

Available traffic assignment programs that build minimum time paths between zones were utilized for the determination of zone-to-zone time via the transit system. The general rules required for coding a highway network were followed for this purpose (*5*).

The route schedules for the 7- to 9-a.m. time period for each of the four transit companies operating in the Washington, D.C., area in 1955 were used for the preparation of a transit network. Link lengths were determined by measurement from scaled maps showing actual route locations and times. All connections from zone centroids to the transit stops were coded as having zero distance and time because these excess traveltimes were coded as zone parameters.

The transit times reported in the 1955 O–D showed such great variation that it was impossible to develop general rules for adjusting the tree times obtained from the transit network prepared from schedules. As there was no apparent bias in the use of tree times, O–D times sometimes being less than and sometimes greater than tree times, the schedule times were utilized in this test.

Transit fare matrix

The modal split procedure requires input information that will provide the zone-to-zone cost via transit. It was obvious that if all 400 zones were considered, a matrix having 160,000 entries would be required. Because many zone-to-zone movements have similar costs, larger areas—termed superzones—can usually be used for this representation. For this test, the 68 districts of the 1955 O–D survey were used.

Transit transfer matrix

The excess traveltime for a transit trip includes the time spent transferring between vehicles. Again, as for the fare matrix, a determination of these times for each zone-to-zone movement would have been a most time-consuming task. Many district-to-district transfer times were already available from the work accomplished in the development of the modal split relationships. The coded transit system provided the necessary detail for the computation of the remaining transfer times. Transfer times were determined by tracing the most logical route(s) between pairs of districts and accumulating one-half the headway for each transit route to which a transfer was made. This procedure deviates from that used when the NCTA estimates were made of 1980 transit usage. For 1980, corridors of influence were drawn about the major radial transit lines, and the times to transfer between these corridors were determined and later expanded to zone-to-zone movements by the modal split program.

The results obtained from the work done by TRC in developing the modal split relationships were used as a general control on the procedure. That is, many of the transfer times available were recalculated for checking purposes and to provide a consistent means for developing the remaining transfer times.

During NCTA's calibration of the procedure to estimate 1980 transit usage, it appeared that illogical zonal trip interchanges by transit between areas outside the 10-mile square (see figure 2) had been estimated by the modal split procedure. To eliminate these illogical interchanges, excessively large transfer times had been entered into the transfer matrix for these movements. These excessive times tended to produce a travel-time ratio greater than ten, thereby producing a zero percent transit usage. To conform with this NCTA procedure, certain interchanges between areas outside the 10-mile square were also eliminated in the BPR test of the procedure against the 1955 O–D survey.

Zonal Parameters

Economic status

TRC, in the development of the modal split relationships, calculated the median 1955 income per worker. As travelers' incomes were not reported in the 1955 O–D survey, average district incomes were calculated from 1950 and 1960 census reports and converted to median income per worker. This calculation was made by multiplying the census income per dwelling unit by the ratio of dwelling units per worker from the expanded household data obtained in the 1955 home interview survey (*6*). The results obtained for district incomes are tabulated in volume II of the TRC reports. These reported incomes were used directly for the Public Roads test after they were coded into the five economic status groups.

Parking costs

Average parking costs are required for inclusion in the cost ratio in the modal split procedure. As every trip is considered to have a return portion, only one-half of the parking cost was assessed at the destination end when the cost ratio was calculated. The parking costs for 1980 were developed basically for work trips. To obtain an estimate of parking costs for nonwork trips, costs calculated for work trips were divided by two. Parking costs were assessed only for zero sector destinations and were developed for each zone. The 1955 average automobile parking costs were obtained for each of the nine zero sector districts by determining the weighted average cost of commercial and government parking facilities in each district.

Car occupancy

Car occupancy rates are required by the modal split procedure for two reasons: In calculating automobile costs for each zone-to-zone movement, the sum of the automobile operating costs and one-half of the total daily parking rate was divided by the number of persons per vehicle to obtain an average automobile travel cost per person. Although the modal split program allows the use of car occupancy at the origin and/or destination, destination car occupancy rates were used in the Public Roads test because they were the rates used in the NCTA estimate for 1980. Car occupancy rates also were used to convert the person trip output from the modal split technique to automobile driver trips. As car occupancy rates for 1980 were developed on a district basis, district rates also were used for the test. These rates were developed by TRC by a special run of the 1955 O–D data through available summary programs.

Walking time

Walking times from parking places to destinations were developed on a district basis for the 1980 estimate and for the development of the modal split technique. One minute walking time was used for every district outside the zero sector. For zero sector districts, the district walking times from parking places to destinations, which had been used by TRC for curve development, were used for this test. All walking time was calculated from blocks reported as walked in the travel surveys.

Parking delay time

For curve development, and in the tests with the 1955 O–D data, parking delay times were based on delays of 1 minute at government parking lots and garages and of 2 minutes at public and private lots and garages—weighted averages were used for each zero sector district (*6*). A 1-minute time was used for each district outside the zero sector.

Transit waiting times

For both the curve development with 1955 data and the 1980 application of the procedure, transit waiting times were determined from a map showing all transit routes, vehicle headways, and transfer points. In general, one-half the average headway of the transit facility serving a zone was used where only one facility served the area. The modal split program allows only one figure as the average transit waiting time for the passengers of each zone. For this reason, the zonal transit waiting times were calculated by utilizing the peak hour transit network, which had been coded for this test. These zonal estimates were averaged by district for comparison with the district waiting times used by TRC for curve development; when different, adjustments were made to the zonal estimates.

Transit walking time

Average walking times to and from transit stops in each zone were determined in a systematic manner based on empirical formulas in the initial development of the modal split technique (*6*). For the 1980 estimates of transit usage, the walking times were obtained from an analysis of the relationship between the location of each zone centroid and the transit routes available in the zone. This latter relationship was used for developing walking times by zone for this test. Again, the average transit walking time obtained for a district for use in this test was compared with the district times used in the development of the modal split tech-

Table 1.—Results of modal split procedure applied to entire study area

Trip	1955 O–D data	BPR test estimates	Percentage point difference	Percent trip difference
Work:				
All modes	389,301.0	389,301.0		
Transit passenger	131,066.2	123,305.2		−5.9
Percent transit usage	33.7	31.8	−1.9	
Nonwork:				
All modes	30,294.0	30,294.0		
Transit passenger	5,614.4	7,040.8		+25.4
Percent transit usage	18.5	23.0	+4.5	
TOTAL:				
All modes	419,595.0	419,595.0		
Transit passengers	136,680.6	130,346.0		−4.6
Percent transit usage	32.6	31.1	−1.5	

Table 2.—Modal split to zero sector (excludes intrazero sector trips)

Trip	1955 O–D data	BPR test estimates	Percentage point difference	Percent trip difference
Work:				
All modes	171,329.9	171,329.9		
Transit passenger	76,723.9	75,678.0		−1.36
Percent transit usage	44.8	44.2	−0.6	
Nonwork:				
All modes	5,141.0	5,141.0		
Transit passenger	2,203.5	2,410.0		+9.37
Percent transit usage	42.9	46.9	+4.0	
TOTAL:				
All modes	176,470.9	176,470.9		
Transit passengers	78,927.4	78,088.0		−1.06
Percent transit usage	44.7	44.2	−0.5	

Table 3.—Modal split to nonzero sector destination

Trip	1955 O–D data	BPR test estimates	Percentage point difference	Percent trip difference
Work:				
All modes	203,942.8	203,942.8		
Transit passenger	45,529.4	38,459.2		−15.53
Percent transit usage	22.3	18.9	−3.4	
Nonwork:				
All modes	22,776.6	22,776.6		
Transit passenger	2,607.1	3,423.9		+31.33
Percent transit usage	11.4	15.0	+3.6	
TOTAL:				
All modes	226,719.4	226,719.4		
Transit passengers	48,136.5	41,883.1		−12.99
Percent transit usage	21.2	18.5	−2.7	

nique. Adjustments were made when necessary. These transit walking times were applied at both the origin and destination of a trip.

Single Parameters

The modal split procedure requires that certain constants be specified. The constants used for the test are, as follows: Cost per gallon of gasoline, 29.5 cents: cost of oil change and lubrication, $2.85; miles between oil change and lubrication, 1,000. These are the same constants used for curve development. The coefficients used in the equations (*6*) for the calculation of car operating costs are the same as those used for development of the modal split technique. The modal split relationships used by Public Roads were the same relationships used for NCTA's 1980 estimates. Other than as heretofore explained, all parameters used in the Public Roads test were the same as those used for the 1980 transit usage estimates (*6*).

Results

The data described in the foregoing paragraphs were used as input to the modal split computer program to obtain an estimate of zone-to-zone transit usage. This estimate was obtained by applying the modal split technique to the total person trip file. These estimates of zone-to-zone transit usage could then be compared with the transit usage observed from the 1955 O–D survey. These comparisons are shown in tables 1 through 8.

Modal split to entire area—table 1

The number of transit work trips for the entire area obtained by application of the modal split procedure to the 1955 O–D survey data was 5.9 percent less than the actual number reported in the survey. The modal splits differed by 1.9 percentage points: Transit work trips reported in the 1955 O–D survey were 33.7 percent of total usage and those estimated by application of the procedure were 31.8 percent. A greater difference between the actual 1955 data and the test data was obtained for nonwork trips, but this difference was an overestimate of 25.4 percent by application of the modal split technique. Total trips via transit were underestimated by 4.6 percent—a 1.5 percentage point difference.

Estimates of citywide transit trips based on relationships developed from actual data would be expected to correspond closely to the base data. In this test the procedure reproduced the base data reasonably well for work trips. However, the modal split technique produced a nonwork trip figure showing a larger difference from the actual data than should be acceptable on a citywide basis. This was true although the volume of nonwork trips was relatively small in comparison to the number of work trips. The difference in test results and actual data for nonwork trips might have been caused by the following listed conditions. (1) The nonwork trip curves were developed from data for offpeak traffic hours but applied to the peak hour traffic. (2) Input parameters were highly oriented to work trips. The only input variable changed for the nonwork trips was the parking cost, and this was estimated to be equal to one-half of the parking costs for work trips. Other variables such as car occupancy, walking times, parking delay time, etc., generally were developed for peak hour work trips but were applied unchanged to nonwork trips.

Modal split to zero sector—table 2

The test with the modal split to the zero sector reproduced the 1955 O–D transit usage remarkably well for work trips. The nonwork trip test estimate differs from the O–D transit usage by +9.37 percent. From the data in tables 1 and 2, the following conclusions might be drawn:

As the modal split relationships for the test were developed almost entirely from zero sector oriented trips, it would be expected that such trips would have been reproduced more accurately than nonzero sector destination trips. Only about 100 nonzero sector interchanges were considered when the modal split relationships were developed.

The modal split technique was more accurate for estimating work trips than nonwork trips because the work-trip input data were developed from and applied to peak-hour traffic conditions; whereas, the nonwork trip data were applied to the peak-hour traffic but had been developed from nonpeak-hour traffic conditions.

Application of nonwork relationships to the peak traffic hours implied that relationships established for any period of the day could be used satisfactorily with the modal split technique. The results obtained in the test against the 1955 O–D data did not substantiate this implication. Therefore, further stratification of data for the input curves by destination—downtown separate from nondowntown—and time of day may be warranted.

(*Continued on page 13*)

Line	State	Width inches[1]	Height ft.-in.	Length-feet[2]					Number of towed units[3]			Axle load-pounds				Operating tire inflation pressure pounds per sq. in.	Pounds per engine net-horsepower delivered to clutch or equivalent
				Single unit								Single		Tandem			
				Truck	Bus	Semi-trailer or trailer	Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance		
1	Alabama	96	13-6	40	40	N5	55	NP	1	NP	NP	18,000	19,800	36,000	39,600	N5	N5
2	Alaska	96	12-6	35	[6]40	[7]40	60	60	1	1	2	18,000		32,000		NS	NS
3	Arizona	96	13-6	40	40	N5	65	65	1	1	2	18,000		32,000		NS	NS
4	Arkansas	96	13-6	40	40	NS	55	55	1	1	NP	[8]18,000		32,000		N5	NS
5	California	96	13-6	35	[9]35	[7]40	60	65	NR	NR	NR	18,000		32,000		N5	NS
6	Colorado	[10]96	[11]13-6	35	40	NR	60	[12]60	1	2	2	18,000		36,000		N5	NS
7	Connecticut	102	12-6	50	50	NR	50	NP	1	NP	NP	22,400	22,848	36,000	36,720	N5	NS
8	Delaware	96	13-6	40	42	40	55	60	1	1	NP	20,000		36,000		NS	NS
9	Florida	96	13-6	[13]35	40	[14] NS	55	55	1	1	NP	20,000	22,000	40,000	44,000	N5	NS
10	Georgia	96	13-6	[15]+39	[15]+45	NR	50	50	1	1	NP	18,000	20,340	36,000	40,680	NS	NS
11	Hawaii	108	13-0	40	40	NR	55	65	1	1	2	24,000		32,000		N5	NS
12	Idaho	[10]96	14-0	[16]35	[16]35	NR	[19]60	65	1	1	2	[20]18,000		[20]32,000		N5	NS
13	Illinois	96	13-6	42	42	42	[21]55	[22]60	1	1	2	[23]18,000		32,000		NS	NS
14	Indiana	[10]96	13-6	36	40	NR	55	[21]55	1	1	2	[25]18,000	[25]19,000	[25]32,000	[25]33,000	NS	NS
15	Iowa	96	13-6	35	[6]40	[7]N5	[26]55	[26]55	1	1	2	18,000	18,540	32,000	32,960	N5	NS
16	Kansas	96	13-6	35	[6]40	N5	50	50	1	1	NP	18,000		32,000		NS	NS
17	Kentucky	96	[11]13-6	[27]35	[27]35	NR	[28]50	[28]50	1	1	NP	18,000	[29]18,900	[29]32,000	33,600	N5	NS
18	Louisiana	96	13-6	35	[6]40	NR	55	60	1	1	NP	18,000		32,000		N5	NS
19	Maine	102	[31]13-6	55	55	NR	55	55	1	1	NP	[31]22,000		[31]32,000		N5	NS
20	Maryland	[32]96	[33]12-6	55	55	NR	55	[34]55	NR	NR	NR	22,400		[35]40,000		N5	NS
21	Massachusetts	96	N5	35	[6]40	NR	50	NP	1	NP	NP	22,400		36,000		NS	NS
22	Michigan	96	13-6	35	40	40	55	55	1	1	2	[37]18,000		[38]32,000		NS	NS
23	Minnesota	96	13-6	40	40	40	50	50	1	1	NP	18,000		32,000		N5	NS
24	Mississippi	96	13-6	35	40	NR	55	55	1	1	NP	18,000		28,650	[22]32,000	NS	NS
25	Missouri	96	12-6	35	40	NR	50	50	1	1	2	18,000		32,000		NS	NS
26	Montana	[10]96	13-6	35	40	NR	60	60	1	1	2	18,000		32,000		NS	NS
27	Nebraska	96	13-6	40	40	[39]NR	60	60	1	1	2	18,000	18,900	32,000	33,600	N5	NS
28	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	N5	NS
29	New Hampshire	96	13-6	35	[22]40	NR	55	55	NR	NR	NR	22,400		36,000		N5	NS
30	New Jersey	[41]96	[41]13-6	35	[42]35	[7]40	53	50	1	1	NP	22,400	23,520	32,000	33,600	N5	NS
31	New Mexico	[43]96	13-6	40	40	NR	65	65	1	1	2	21,600		34,320		N5	NS
32	New York	96	[33]13-0	35	[44]35	[7]NR	50	50	1	1	NP	22,400		36,000		NS	NS
33	North Carolina	96	13-6	35	[6]40	NR	55	55	1	1	NP	18,000	19,000	36,000	38,000	NS	NS
34	North Dakota	[41]96	[41]13-6	[13]35	[6]40	NR	60	60	1	1	2	18,000		32,000		N5	NS
35	Ohio	96	13-6	35	[6]40	[7]40	55	60	1	NR	NR	19,000		31,500		NS	NS
36	Oklahoma	96	13-6	35	45	NR	[45]50	[45]50	1	1	NP	18,000		32,000		N5	NS
37	Oregon	96	[11]13-6	35	[22]40	[22]40	[21]55	[22]65	1	1	[22]2	[46]18,000		[46]32,000		N5	NS
38	Pennsylvania	96	[33]12-6	35	40	40	[48]50	[34]50	1	1	NP	22,400	23,072	36,000	37,080	N5	NS
39	Rhode Island	102	12-6	40	40	40	50	50	1	1	NP	22,400		NS		NS	NS
40	South Carolina	96	13-6	40	[6]40	NR	55	[54]60	1	1	NP	20,000		[55]32,000		NS	NS
41	South Dakota	96	13-6	35	40	NR	[22]65	[22]65	1	1	2	18,000		32,000		NS	NS
42	Tennessee	96	13-6	35	40	[7]NS	50	50	1	[56]1	NP	18,000		32,000		NS	NS
43	Texas	96	13-6	35	40	NS	50	50	1	1	NP	18,000		32,000		N5	NS
44	Utah	96	14-0	45	45	45	60	60	NR	NR	NR	18,000		33,000		NS	NS
45	Vermont	96	13-6	50	50	NS	55	55	1	1	NP	22,400	23,520	[57]36,000		NS	NS
46	Virginia	96	13-6	35	40	NR	50	50	1	1	NP	18,000		[58]32,000		N5	NS
47	Washington	96	13-6	35	[6]40	40	[19]60	65	1	1	2	18,000		32,000		N5	NS
48	West Virginia	96	[33]12-6	35	[6]40	35	50	50	1	1	NP	18,000	18,900	32,000	33,600	NS	NS
49	Wisconsin	96	13-6	35	40	35	55	55	1	1	NP	18,000	[60]19,500	30,400	32,000	NS	NS
50	Wyoming	96	13-6	40	40	NR	65	65	1	1	2	18,000		32,000	[62]36,000	NS	NS
51	District of Columbia	96	12-6	40	40	NS	50	50	1	1	NP	22,000		38,000		N5	NS
52	Puerto Rico	96	12-6	35	40	N5	50	50	1	1	NP	NS		N5		NS	NS
	AASHO Policy	102	13-6	40	40	40	55	65	1	1	2	20,000		32,000		95	400
Number of States	Higher	1	4	7	10	35	14	1	5	7	6	16		26		52	52
	Same	3	37	12	37	7	20	9	47	42	18	2		25		0	0
	Lower	48	11	33	5	10	18	42	0	3	28	34		1		0	0

NP–Not permitted. NR–Not restricted. NS–Not specified.

[1] Various exceptions for farm and construction equipment, public utility vehicles, house trailers, urban, suburban, and school buses, haulage of agricultural and forest products, at wheels of vehicles for safety accessories, on designated highways, and as administratively authorized.

[2] Various exceptions for utility vehicles and loads, house trailers and mobile homes.

[3] When not specified, limited to number possible in practical combinations within permitted length limits, various exceptions for farm tractors, mobile homes, etc.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits

(1) Minimum front overhang of 3 feet, minimum spacing from first to second axle of truck tractor 8 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerance as applicable.

[6] Less than three axles 35 feet.

[7] Trailer 35 feet

[8] Steering axle 12,000 pounds.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 45 feet without restriction.

[10] Buses 102 inches on highways of surfaced width at least 20 feet or otherwise as administratively authorized.

[11] On class AA, or designated highways, 12 ft. 6 in. on other highways, log and lumber trucks limited to 12 ft. 6 in. on all highways in Oregon.

[12] Except 3-unit combinations may use up to 65 ft. combinations on certain highways designated by the Department of Highways.

[13] Three-axle vehicles 40 feet.

[14] Two-axle trailer 35 feet, three-axle trailer 40 feet.

[15] Truck 39.55 feet, bus 45.20 feet.

[16] 63,280 pounds maximum, except on roads under Ru

[17] 700 (L+40) when L is 18' or less, 800 (L+40) wh span of 20' or over.

[18] On designated highways 40 feet.

[19] Auto transports on designated highways 65 feet.

[20] Special limits for vehicles hauling timber and timb livestock, single axle 18,900 pounds, tandem axle 37,800 pounds maximum at 21-foot axle spacing, vehicle with 5 or

[21] 60 ft. in special cases: Illinois, auto transports trailers on designated major routes.

[22] On designated highways only.

[23] On designated highways, 16,000 pounds on other

[24] Axle spacing 44 feet or more, otherwise 72,000 p

[25] On designated highways, single axle 22,400 poun of weight under one or more limitations of axle load and g axle.

[26] Auto and boat transports and three-unit combinati wise 50 feet.

[27] On designated highways, trucks 26.5 feet and bu

[28] Class AA highways, 45 feet on other highways.

[29] Class AA highways only.

[30] Maximum gross weight on Class A highways 42,0

[31] Including load 14 feet, various exceptions for ve

[32] Vehicles loaded with tobacco hogsheads–103 inc

[33] Auto transports 13 feet 6 inches, Maryland also flat glass.

[34] Exception for poles, pilings, structural units, ro

[35] Less than 48-inch spacing, 36,000 pounds.

[36] Subject to axle and tabular limits.

[37] Single axle spaced less than 9 feet from nearest

ight limit			Specified maximum gross weight-pounds[4]						Practical maximum gross weight-pounds[5]						
	Applicable to:		Truck		Truck-tractor semitrailer				Truck		Truck-tractor semitrailer				
	Any group of axles	Total wheelbase only	2-axle	3 axle	3-axle	4-axle	5-axle	Other combination	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	Line
		X					73, 280		27, 800	47, 600	47, 600	67, 400	73, 280	NP	1
	Under 18'	Over 18'	36, 000	50, 000	50, 000	72, 000	76, 800	76, 800	26, 000	40, 000	44, 000	58, 000	72, 000	76, 800	2
	Under 18'	Over 18'							26, 000	40, 000	44, 000	58, 000	72, 000	76, 800	3
	Under 18'	Over 18'							26, 000	40, 000	44, 000	58, 000	72, 000	73, 280	4
									26, 000	40, 000	44, 000	58, 000	72, 000	76, 800	5
n.		X							26, 000	44, 000	44, 000	62, 000	76, 000	76, 000	6
p.			32, 000	53, 800	53, 800	67, 400	73, 000	NP	30, 848	44, 720	53, 800	67, 400	73, 000	NP	7
	X		30, 000	46, 000	48, 000				28, 000	44, 000	48, 000	64, 000	73, 280	73, 280	8
		X							30, 000	52, 000	52, 000	73, 271	73, 271	73, 271	9
								63, 280	28, 340	48, 680	48, 680	63, 280	63, 280	63, 280	10
	X								32, 000	40, 000	56, 000	64, 000	72, 000	80, 000	11
	X								26, 000	40, 000	44, 000	58, 000	73, 280	76, 800	12
p.			[1] 36, 000	50, 000	50, 000	64, 000	[24] 73, 280	[24] 73, 280	26, 000	40, 000	44, 000	58, 000	72, 000	73, 280	13
p.			36, 000	50, 000	54, 000			72, 000	27, 000	41, 000	45, 000	59, 000	[25] 73, 000	[25] 73, 000	14
	X								26, 540	40, 960	45, 080	59, 500	73, 280	73, 280	15
	X								26, 000	40, 000	44, 000	58, 000	72, 000	73, 280	16
p [30]			27, 000	42, 000	42, 000	59, 640	73, 280	NP	27, 000	42, 000	42, 000	59, 640	73, 280	73, 280	17
									26, 000	40, 000	44, 000	58, 000	72, 000	76, 000	18
	X		32, 000	[31] 51, 800	51, 800	62, 050	73, 280	73, 280	30, 000	40, 000	51, 800	62, 000	72, 000	73, 280	19
		X			[36] 65, 000	[36] 73, 280	[36] 73, 280	[36] 73, 280	30, 400	48, 000	52, 800	70, 400	73, 280	73, 280	20
			[36] 46, 000	[36] 60, 000	[36] 60, 000	[36] 73, 000	[36] 73, 000	NP	30, 400	44, 000	52, 800	66, 400	73, 000	NP	21
									26, 000	[22] 40, 000	44, 000	[22] 58, 000	[22] 66, 000	[22] 102, 000	22
	X						73, 280	73, 280	26, 000	40, 000	44, 000	58, 000	72, 000	73, 280	23
	X								26, 000	[22] 40, 000	44, 000	58, 000	[22] 64, 650	[22] 73, 280	24
	X								26, 000	40, 000	44, 000	58, 000	[22] 72, 000	[22] 73, 280	25
	Under 18'	Over 18'							26, 000	40, 000	44, 000	58, 000	72, 000	76, 000	26
	X		36, 000	54, 000	54, 000	71, 146	71, 146	71, 146	26, 780	41, 200	45, 320	59, 740	73, 280	73, 280	27
	Under 18'	Over 18'							26, 900	41, 600	45, 800	60, 500	75, 200	76, 800	28
		X	33, 400	[40] 55, 000	[40] 55, 000	66, 400	73, 280	73, 280	30, 400	44, 000	52, 800	66, 400	73, 280	73, 280	29
									31, 520	41, 600	55, 040	65, 120	73, 280	73, 280	30
	Under 18'	Over 18'							29, 600	42, 320	51, 200	63, 920	76, 640	86, 400	31
		X					71, 000	71, 000	30, 400	44, 000	52, 800	66, 400	71, 000	71, 000	32
			31, 500	49, 875	49, 875	67, 200	73, 280	73, 280	27, 000	46, 000	46, 000	65, 000	73, 280	73, 280	33
	Under 18'	Over 18'							26, 000	40, 000	44, 000	58, 000	72, 000	73, 280	34
		X							27, 000	39, 500	46, 000	58, 500	71, 000	78, 000	35
	X								26, 000	40, 000	44, 000	58, 000	72, 000	73, 280	36
	Under 18'	Over 18'					[47] 76, 000	[47] 76, 000	26, 000	40, 000	44, 000	58, 000	72, 000	[47] 76, 000	37
			33, 000	47, 000	50, 000	60, 000	71, 145	71, 145	31, 072	45, 080	51, 500	61, 800	73, 280	73, 280	38
			[50] 36, 000	[51] 44, 000	[52] 53, 800	[53] 67, 400	73, 280	88, 000	30, 400	44, 000	53, 600	67, 400	73, 280	88, 000	39
			32, 000	46, 000	50, 000	65, 000	73, 280	73, 280	28, 000	40, 000	48, 000	60, 000	72, 000	73, 280	40
	X								26, 000	40, 000	44, 000	56, 000	72, 000	73, 280	41
		X		44, 000	48, 000	62, 000	73, 280		26, 000	40, 000	44, 000	58, 000	72, 000	43, 500	42
	X								26, 000	40, 000	44, 000	58, 000	72, 000	72, 000	43
	X		36, 000	51, 000	54, 000	69, 000	79, 900	79, 900	26, 000	41, 000	44, 000	59, 000	74, 000	79, 900	44
	X						73, 280	73, 280	31, 520	44, 000	55, 000	66, 400	73, 280	73, 280	45
	X						70, 000	70, 000	26, 000	40, 000	44, 000	60, 000	70, 000	70, 000	46
	Under 18'	Over 18'	28, 000	36, 000	46, 000	60, 000	68, 000	72, 000	26, 000	36, 000	44, 000	60, 000	68, 000	72, 000	47
	X					[59] 70, 000	[59] 70, 000	[59] 70, 000	26, 900	41, 600	45, 800	60, 500	73, 280	73, 280	48
	X								27, 500	40, 000	47, 000	59, 500	73, 000	73, 000	49
	X								26, 000	44, 000	44, 000	62, 000	73, 950	73, 950	50
	X					70, 000	70, 000	70, 000	30, 000	46, 000	52, 000	68, 000	70, 000	70, 000	51
b.															52
	X								28, 000	40, 000	48, 000	60, 000	72, 000	86, 500	
5									15	25	15	22	24	2	
32	20	17							2	24	2	3	19	0	
15									34	2	34	26	8	49	

[38] On designated highways only and limited to one tandem axle in combination, otherwise 26,000 pounds.
[39] Trailer 40 feet.
[40] On Interstate System 47,500 pounds.
[41] Vehicles in excess may be operated under special permit obtained in advance, in New Jersey from the Department of Motor Vehicles, in North Dakota, from State Highway Truck Regulatory Department.
[42] Or as prescribed by P.U.C.
[43] On designated highways 102 inches. Body restricted to 96", additional 6" for tires only.
[44] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.
[45] Auto transports, oil field equipment, by special permit only, 60 feet.
[46] Logging vehicles permitted 7-foot wheelbase tolerance, 19,000-single axle, 34,000-pounds tandem axle.
[47] Governs gross weight permitted on highways designated by resolution of State highway commission.
[48] Where truck-tractor was properly registered in Pennsylvania as of December 31, 1961, 55 feet.
[49] Single unit truck with 4 axle permitted 60,000 pounds.
[50] Axles spaced less than 6 feet 32,000 pounds, less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.
[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds, less than 20 feet 44,000 pounds, 20 feet or more governed by axle limit.
[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds, not less than 27 feet 53,800 pounds.
[53] Legal limit 67,400 pounds, axle spacing 27 feet or more.
[54] House trailers only, otherwise 55 feet.
[55] On Interstate System, 36,000 pounds on other roads.
[56] Limited to 3,500 pounds.
[57] Three-axle tandem 42,700 pounds.
[58] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle
[59] Only on certain highways, or portions thereof, designated by State Roads Commissioner, and consistent with Congressional action.
[60] Axle load 21,000 pounds on 2-axle trucks hauling peeled or unpeeled forest products cut crosswise or transporting milk from farm to market but not over Interstate System.
[61] On Class A highways. All axles of a vehicle or combination—73,000 pounds maximum. Wheel, axle, axle group and gross vehicle weights on Class B highways are 60% of weights including tolerance authorized for Class A highways.
[62] Based on ruling of Attorney General.

6,000 pounds maximum.
8', 900(L + 40) on highways having no structures with

entrates, aggregates, and agricultural products including
able" vehicle with 3 or 4 axles permitted 66,000
79,000 pounds maximum at 43-foot axle spacing.
outling house trailers only, Oregon, truck tractor semi-

pounds; tolerance of 1,000 pounds on total of all excesses
g upon the placing of 9000# on the front or steering

on highways with surface width 22 feet or more, other-
ghways.

highways 30,000 pounds.
roducts and construction materials.

far vehicles loaded with hay or straw, or carrying

tted 70 feet.

pounds.

(Continued from page 9)

Table 4.—Modal split from each corridor to CBD

Sector (origin)	All modes	1955 O–D data		BPR test estimates		Percentage point difference	Percent trip difference
		Transit trips	Percent transit usage	Transit trips	Percent transit usage		
A—WORK TRIPS							
			Percent		*Percent*	*Percent*	*Percent*
0	14,028.3	8,807.9	62.8	9,168.0	65.4	+2.6	+4.1
1	9,529.9	5,114.1	53.7	4,407.3	46.2	−7.5	−13.8
2	25,231.3	9,209.5	36.5	8,428.8	33.4	−3.1	−8.5
3	45,720.9	23,748.7	51.9	25,337.6	55.4	+3.5	+6.7
4	24,291.8	9,948.0	41.0	10,948.0	45.1	+4.1	+10.1
5	12,348.1	6,199.9	50.2	6,484.1	52.5	+2.3	+4.6
6	21,119.2	10,272.4	48.6	10,569.9	50.0	+1.4	+2.9
7	15,750.7	6,315.9	40.1	4,922.6	31.3	−8.8	−22.1
8	17,338.0	5,915.4	34.1	4,579.7	26.4	−7.7	−22.6
B—NONWORK TRIPS							
0	2,376.4	803.8	33.8	1,206.9	50.8	+17.0	+50.2
1	240.7	67.2	27.9	91.1	37.8	+9.9	+35.6
2	601.2	108.8	18.1	142.6	23.7	+5.6	+31.1
3	1,663.8	821.1	49.4	969.8	58.3	+8.9	+18.1
4	626.8	326.2	52.0	354.8	56.6	+4.6	+8.9
5	569.5	369.5	64.9	306.6	53.8	−11.1	−17.0
6	712.4	339.8	47.6	355.6	49.9	+2.3	+4.7
7	368.8	105.5	28.6	140.8	38.2	+9.6	+33.0
8	357.8	65.4	18.3	48.7	13.6	−4.7	−26.0
C—WORK AND NONWORK TRIPS							
0	16,404.7	9,611.7	58.6	10,374.9	63.2	+4.6	+7.9
1	9,770.6	5,181.3	53.0	4,498.4	46.0	−7.0	−13.2
2	25,832.5	9,318.3	36.1	8,571.4	33.2	−2.9	−8.0
3	47,384.7	24,569.8	51.9	26,307.4	55.5	+3.6	+7.1
4	24,918.6	10,274.2	41.2	11,302.8	45.4	+4.2	+10.0
5	12,917.6	6,569.4	50.9	6,790.7	52.6	+1.8	+3.4
6	21,831.6	10,612.2	48.6	10,925.5	50.0	+1.4	+3.0
7	16,119.5	6,421.4	39.8	5,063.4	31.4	−8.4	−21.1
8	17,695.8	5,980.8	33.8	4,628.4	26.2	−7.6	−22.6

Modal split to nonzero sector destinations—table 3

Results of the analysis of test results obtained with the modal split technique for nonzero sector destinations strengthens the conclusions reached from data shown in tables 1 and 2. That is, estimates for zero sector trips were more reliable than estimates for nonzero sector trips.

Modal split from each corridor to the CBD—table 4

The amount of transit travel along individual corridors or sectors to the downtown area shows a greater dispersion than does the areawide test estimate. The sectors considered are shown in figure 2. An analysis of the test results indicated a geographical bias in the estimates of modal split for work trips. Sectors 1, 2, 7, and 8 are all west of a line set by Rock Creek Park and the Potomac River. For these sectors the estimates of transit trips were about 16 percent less than shown by the O–D data. For all sectors east of this line a 6-percent overestimate of transit trips was obtained with the modal split technique.

The nonwork trip dispersion was far greater than dispersion for work trips in this test, but the test result for the nonwork trips does not indicate the same geographical bias as the work trip estimate. The total estimate (work and nonwork) shows the same geographical bias as the estimate for the work trips because the number of nonwork trips was relatively small. Knowledge of the weakness of the modal split in providing reliable estimates for transit travel along corridors to the CBD is particularly important because it is these corridor movements that are used for system planning when transit usage must be estimated for some future period.

Modal split between survey area sectors—tables 5 and 6

The data on the modal split between survey area sectors were used in the development of the previously discussed tables. The test results for dispersion in trip estimation between sectors did not follow any clearly observable pattern. The tables are presented for reference purposes.

Analysis of district-to-district transit trips—tables 7 and 8

The zone-to-zone transit trip files for both the test estimated and O–D trips were summarized into district-to-district movements for the 68 districts within the 1955 O–D survey area. These movements were classified into volume groups, in accordance with the O–D survey movements, and compared. The statistical procedure used was the root-mean-square error analysis (7):

$$\text{RMS error} = \sqrt{\frac{\sum_{i=1}^{n} (\text{O-D} - \text{model})^2}{n}}$$

Where,

O–D = movement between pair of districts from O–D survey for a specified volume group.

model = movement between the same pair of districts from modal split procedure.

n = number of O–D pairs in volume group.

The percent root-mean-square error, equal to the root-mean-square error divided by the mean O–D volume for the volume group, was used as the measure of comparison to relate the accuracy of the test results obtained from the model to the O–D survey data and also to state the accuracy of the O–D survey volumes. The Washington, D.C., 1955 survey was made with an average dwelling unit sample of 6.2 percent. From the results of previous research (7), the percent RMS errors to be expected from such a sample were known. For both work and nonwork trips, the percent RMS error between the estimate of transit trips produced by the modal split technique and the 1955 O–D survey estimate of these trips is less than the error expected in the expanded 6.2 percent sample of survey volumes.

If the comparison had shown a greater error between O–D and modal split procedure transit trips than was shown between the O–D trips and actual trips, it could be stated that the estimate of actual transit trips made with the model was worse than the estimate made by the O–D survey. However, the results obtained only indicate that the variation between the O–D and model estimates was less than the variation in the survey. On this basis, the modal split procedure would appear to give reasonable results.

Trip length frequency

Figure 3 presents a comparison of trip length frequency distributions from the O–D survey and modal split estimates for work trips. The modal split procedure produced an average trip length estimate just slightly longer than that produced by the O–D survey, 24.6 and 23.5 minutes, respectively. The trip length distributions were in reasonably close agreement. A cumulative frequency by 10-minute increments is shown in table 9.

Sensitivity Tests

Sensitivity tests also were developed by Public Roads to assess the reliability of the procedure when input variables were changed. The effectiveness of the modal split procedure to realistically project changes that might occur in the input variables over a period of time were not assessed as part of the Public Roads evaluation previously described herein. As the 1955 O–D survey data were a primary source for development of the procedure, a test performed on the same data would

basically indicate whether the procedure would recreate the information from which it had been developed. The sensitivity tests were made because the change in the modal split that would be predicted if, for example, the headways on the transit system were halved, had not been measured, in the test against the 1955 O–D.

The tests made with the 1955 O–D data and the sensitivity tests were made concurrently, this necessitated use of the 1980 NCTA estimates as a basis for the sensitivity tests. Data developed by the NCTA for 1980 were used for all sensitivity tests, except for the changes made in items selected for testing purposes. Only work trips were evaluated. This evaluation was made by using person trip estimates from 1980 land-use plan B (corridor plan). Inputs changed were: Transit fare, median income, automobile parking delay and walking time, transit waiting and transfer times, parking costs, transit vehicle times, highway vehicle times, parking costs in combination with automobile excess times, and highway times in combination with parking costs and automobile trip walking times. In these sensitivity analyses, feedback between the automobile and transit systems and the estimates of the usage of the systems was not considered. A complete account of these tests and the results is presented in the following paragraphs.

Procedure

The modal split computer program is designed to permit changes in input data with a minimum of effort. Certain parameters may be varied by changing a single constant on an input punch card, others require substitution of one input tape. As summarized, 13 runs of the model were made Initially only one parameter was changed per run. In two of the last runs, several parameters were changed so that joint effects could be analyzed. Although the changes were not necessarily intended to reflect realistic changes in system operation, they were made so that justifiable conclusions could be drawn as to the effect of the change in a particular parameter on the estimated modal split.

Base—NCTA run 23

NCTA run 23 was used throughout the sensitivity analysis as a base for comparison purposes. It was the final run for the 1980 NCTA plan B (corridor plan) morning peak traffic hour (7:30-8:30) work trips. The output data were NCTA's final estimate for this land-use plan and its recommended transportation system.

Sensitivity test *A*

For sensitivity test *A*, a 15-cent fare increase was applied to each zonal interchange. This change was accomplished by changing the transit fare matrix and rerunning the model; NCTA run 23 data were used for all other inputs.

Sensitivity test *B*

In sensitivity test *B*, the 1980 zonal incomes were factored by 1.5. This change effectively raised the median economic status areawide to 50 percent above the 1960 level. Because the 1980 income distribution is related to the 1980 land-use plan, the change (1960–1980) was not necessarily 50 percent for each zone.

Table 5.—Modal split between each of survey area sectors—work trips

Sector		All modes	1955 O–D data		BPR test estimates		Percentage point difference	Percent trip difference
Origin	Destination		Transit trips	Percent transit usage	Transit trips	Percent transit usage		
				Percent		*Percent*	*Percent*	*Percent*
0	0	14,028.3	8,807.9	62.8	9,168.0	65.4	+ 2.6	+ 4.1
0	1	1,304.7	803.4	61.6	483.1	37.0	−24.6	−39.8
0	2	2,704.8	1,611.8	59.6	965.1	35.7	−23.9	−40.1
0	3	3,408.6	1,956.1	57.4	1,491.0	43.7	−13.7	−23.8
0	4	2,151.4	1,163.8	54.1	818.8	38.1	−16.0	−29.6
0	5	1,158.9	501.2	43.2	434.5	37.5	− 5.7	−13.4
0	6	2,136.0	1,089.2	51.0	644.4	30.2	−20.8	−40.8
0	7	3,738.3	2,026.4	54.2	1,435.2	38.4	−15.8	−29.2
0	8	1,043.0	441.1	42.3	430.1	41.2	− 1.1	− 2.5
1	0	9,529.9	5,114.1	53.7	4,407.3	46.2	− 7.5	−13.8
1	1	1,455.6	149.9	10.3	284.2	19.5	+ 9.2	+89.4
1	2	1,492.4	156.0	10.5	126.8	8.5	− 2.0	−18.6
1	3	881.7	209.7	23.8	215.5	24.4	+ 0.6	+ 2.9
1	4	692.8	125.2	18.1	180.5	26.1	+ 8.0	+43.9
1	5	281.2	71.2	25.3	82.9	29.5	+ 4.2	+16.8
1	6	379.1	36.9	9.7	70.8	18.7	+ 9.0	+92.1
1	7	1,275.4	190.8	15.0	194.7	15.3	+ 0.3	+ 2.1
1	8	181.8	0.0	0.0	3.3	1.8	+ 1.8	--------
2	0	25,231.3	9,209.5	36.5	8,428.8	33.4	− 3.1	− 8.5
2	1	1,512.5	221.1	14.6	86.4	5.7	− 8.9	−61.0
2	2	8,518.3	340.8	40.0	56.7	0.7	−39.3	−83.3
2	3	4,185.9	119.0	2.8	137.4	3.3	+ 0.5	+15.1
2	4	1,650.0	10.3	0.6	112.9	6.8	+ 6.2	+1000.0
2	5	445.2	74.0	16.6	35.5	8.0	− 8.6	−51.4
2	6	1,117.9	148.4	13.3	124.1	11.1	− 2.2	−16.2
2	7	2,795.7	378.9	13.6	442.5	15.8	+ 2.2	+16.9
2	8	193.8	0.0	0.0	6.5	3.4	+ 3.4	--------
3	0	45,720.9	23,748.7	51.9	25,337.6	55.4	+ 3.5	+ 6.7
3	1	2,420.4	875.2	36.2	676.8	28.0	− 8.2	−22.6
3	2	58,961.1	1,670.8	2.8	1,125.5	1.9	− 0.9	−32.6
3	3	13,076.2	3,223.1	24.6	2,883.4	22.1	− 2.5	−10.5
3	4	4,628.5	786.3	17.0	803.0	17.3	+ 0.3	+ 2.2
3	5	1,513.1	338.9	22.4	515.7	34.1	+11.7	+52.2
3	6	4,224.9	1,063.0	25.2	1,373.7	32.5	+ 7.3	+29.2
3	7	6,726.1	2,180.9	32.4	2,926.3	43.5	+11.1	+34.2
3	8	951.8	241.3	25.4	263.8	27.7	+ 2.3	+ 9.1
4	0	24,291.8	9,948.0	41.0	10,948.0	45.1	+ 4.1	+10.1
4	1	1,118.1	217 2	19.4	251.0	22.4	+ 3.0	+15.6
4	2	2,412.1	524.4	21.7	524.8	21.8	+ 0.1	+ 0.0
4	3	4,715.7	847.7	18.0	930.4	19.7	+ 1.7	+ 9.8
4	4	10,517.3	1,391.4	13.2	725.2	6.9	− 6.3	−47.9
4	5	2,017.9	274.2	13.6	372.1	18.4	+ 4.8	+35.7
4	6	4,204.0	799.0	19.0	659.3	15.7	− 3.3	−17.5
4	7	3,324.5	582.6	17.5	957.4	28.8	+11.3	+64.3
4	8	392.2	68.8	17.5	107.9	27.5	+10.0	+56.7
5	0	12,348.1	6,199.9	50.2	6,484.1	52.5	+ 2.3	+ 4.6
5	1	789.2	417.0	52.8	273.5	34.7	−18.1	−34.5
5	2	1,255.6	624.0	49.7	391.9	31.2	−18.5	−37.2
5	3	2,468.3	950.6	38.5	1,001.5	40.6	+ 2.1	+ 5.4
5	4	3,378.9	881.0	26.1	742.4	22.0	− 4.1	−15.8
5	5	1,981.7	233.4	11.8	335.6	16.9	+ 5.1	+43.7
5	6	3,960.1	1,153.9	29.1	783.7	19.8	− 9.3	−32.1
5	7	2,528.8	606.6	24.0	931.2	36.8	+12.8	+53.6
5	8	342.5	114.0	33.3	71.7	20.9	−12.4	−36.8
6	0	21,119.2	10,272.4	48.6	10,569.9	50.0	+ 1.4	+ 2.9
6	1	908.2	286.1	31.5	332.0	36.6	+ 5.1	+16.1
6	2	1,689.9	689.2	40.8	616.4	36.5	− 4.3	−10.6
6	3	2,073.1	1,018.7	49.1	904.1	43.6	− 5.5	−11.3
6	4	2,530.9	693.0	27.4	595.2	23.5	− 3.9	−14.1
6	5	1,804.5	469.1	26.0	413.1	22.9	− 3.1	−11.9
6	6	10,869.1	1,545.7	14.2	2,131.9	19.6	+ 5.4	+37.9
6	7	3,623.6	582.9	16.1	1,054.3	29.1	+13.0	+80.8
6	8	353.5	112.0	31.7	65.2	18.4	−13.3	−42.0
7	0	15,750.7	6,315.9	40.1	4,922.6	31.3	− 8.8	−22.1
7	1	553.0	68.4	12.4	53.7	9.7	− 2.7	− 2.7
7	2	876.5	164.5	18.8	71.0	8.1	−10.7	−57.1
7	3	707.6	99.6	14.1	134.2	19.0	+ 4.9	+35.1
7	4	436.5	51.4	11.8	47.7	10.9	− 0.9	− 7.8
7	5	448.0	22.2	5.0	15.8	3.5	− 1.5	−27.0
7	6	1,884.1	107.0	5.7	127.2	6.8	+ 1.1	+18.7
7	7	20,788.9	4,955.0	23.8	2,242.2	10.8	−13.0	−54.8
7	8	2,405.4	158.4	6.6	81.4	3.4	− 3.2	−48.6
8	0	17,338.0	5,915.4	34.1	4,579.7	26.4	− 7.7	−22.6
8	1	911.0	102.4	11.2	19.0	2.1	− 9.1	−81.0
8	2	1,029.1	55.0	5.3	58.5	5.7	+ 0.4	+ 5.4
8	3	738.3	22.2	3.0	62.2	8.4	+ 5.4	+180.2
8	4	447.3	22.7	5.1	41.8	9.3	+ 4.2	+83.7
8	5	175.7	0.0	0.0	4.0	2.3	+ 2.3	--------
8	6	1,447.5	34.0	2.3	55.9	3.9	+ 1.6	+64.7
8	7	11,169.4	1,833.4	16.4	584.1	5.2	−11.2	−68.1
8	8	6,435.4	541.2	8.4	255.1	4.0	− 4.4	−52.8

Sensitivity test *C*

In sensitivity test *C*, 1 minute was added to both the parking delay time at the destination and the time spent walking from the parking space to the ultimate destination. These

two items constitute the denominator (excess time via automobile) in the level of service ratio.

Sensitivity test *D*

The time spent waiting for transit in the origin zone and the time spent transferring between transit vehicles was factored by 1.5 in sensitivity test *D*. This was equivalent to a drastic cutback in transit service. This change had a large effect on the level of service ratio and a lesser effect on the traveltime ratio.

Sensitivity test *E*

Parking costs, which were applied only to zero sector zones, were doubled for sensitivity test *E*.

Sensitivity test *F*

In sensitivity test *F*, the transit vehicle traveltime between all zones was factored by 0.75, this in effect speeded up all transit vehicles.

Sensitivity test *G*

For sensitivity test *G*, the automobile time on the highway system was factored by 0.75, this in effect increased the speed of automobile travel between all zones.

Sensitivity test *H*

Transit fares were doubled for sensitivity test *H*. This run, which parallels sensitivity test *A*, was designed to evaluate the range of application of the cost ratio curves.

Sensitivity test *I*

The excess automobile times for 1980 were replaced in sensitivity test *I*, by the 1955 estimates of these times used for development of the modal split curves.

Sensitivity test *J*

The transit vehicle traveltime was factored by 1.5 in sensitivity test *J*.

Sensitivity test *K*

In sensitivity test *K*, the 1980 estimates of parking costs, parking delay time, and walking time to the ultimate destination from the parking place were replaced by 1955 O–D survey data used for the development of the relationships. This test was designed to determine the effect on transit usage caused by changes in automobile terminal conditions (1955–1980).

Sensitivity test *L*

In sensitivity test *L*, highway traveltimes were factored by 0.75, parking cost by 0.66, and walking time from the automobile parking place by 0.66. This test was designed to evaluate the effect of changes favorable to highway travel.

Sensitivity test *M*

Median incomes (1980) were factored by 1.2 for sensitivity test *M*. This run paralleled sensitivity test *B* and was designed to evaluate the procedure's sensitivity to a modest increase in median incomes.

Table 6.—Modal split between each of survey area sectors nonwork trips

Sector		All modes	1955 O–D data		BPR test estimates		Percentage point difference	Percent trip difference
Origin	Destination		Transit trips	Percent transit usage	Transit trips	Percent transit usage		
				Percent		*Percent*	*Percent*	*Percent*
0	0	2,376.4	803.8	33.8	1,206.9	50.8	+17.0	+50.2
0	1	112.4	0.0	0.0	39.3	35.0	+35.0	-------
0	2	224.6	39.8	17.7	110.2	49.1	+31.4	+176.9
0	3	650.5	372.0	57.2	343.3	52.8	−4.4	−7.7
0	4	251.1	40.3	16.0	146.2	58.2	+42.2	+262.0
0	5	204.2	74.9	36.7	87.9	43.0	+6.3	+17.4
0	6	339.4	75.4	22.2	156.5	46.1	+23.9	+107.6
0	7	199.7	63.8	31.9	152.3	76.3	+44.4	+138.7
0	8	141.6	0.0	0.0	65.1	46.0	+46.0	-------
1	0	240.7	67.2	27.9	91.1	37.8	+9.9	+35.6
1	1	407.7	0.0	0.0	1.2	0.3	+0.3	-------
1	2	258.7	0.0	0.0	8.0	3.1	+3.1	-------
1	3	10.8	0.0	0.0	0.0	0.0	0.0	-------
1	4	47.4	0.0	0.0	0.5	1.0	+1.0	-------
1	5	-------	-------	-------	-------	-------	-------	-------
1	6	-------	-------	-------	-------	-------	-------	-------
1	7	-------	-------	-------	-------	-------	-------	-------
1	8	59.3	0.0	0.0	0.2	0.0	0.0	-------
2	0	601.2	108.8	18.1	142.6	23.7	+5.6	+31.1
2	1	334.7	12.0	3.6	6.4	1.9	−1.7	−46.7
2	2	2,522.3	57.3	2.3	0.0	0.0	−2.3	-------
2	3	524.1	0.0	0.0	0.6	0.0	0.0	-------
2	4	131.5	0.0	0.0	0.0	0.0	0.0	-------
2	5	11.1	0.0	0.0	0.0	0.0	0.0	-------
2	6	10.0	0.0	0.0	0.0	0.0	0.0	-------
2	7	50.8	0.0	0.0	1.2	2.4	+2.4	-------
2	8	44.8	0.0	0.0	0.0	0.0	0.0	-------
3	0	1,663.8	821.1	49.4	969.8	58.3	+8.9	+18.1
3	1	75.2	35.6	47.3	42.6	56.6	+9.3	+19.7
3	2	297.7	0.0	0.0	38.2	12.8	+12.8	-------
3	3	3,113.5	356.0	11.4	399.4	12.8	+1.4	+12.2
3	4	402.6	68.6	17.0	99.8	24.8	+7.8	+45.6
3	5	131.1	76.1	58.0	94.5	72.1	+14.1	+24.2
3	6	204.4	155.6	76.1	66.2	32.4	−43.7	−57.5
3	7	139.3	0.0	0.0	48.5	34.8	+34.8	-------
3	8	11.3	0.0	0.0	0.0	0.0	0.0	0.0
4	0	626.8	326.2	52.0	354.8	56.6	+4.6	+8.9
4	1	9.9	0.0	0.0	0.0	0.0	0.0	-------
4	2	162.3	0.0	0.0	0.0	0.0	0.0	-------
4	3	808.4	92.7	11.5	223.1	27.6	+16.1	+140.8
4	4	1,668.5	57.3	3.4	67.9	4.1	+0.7	+19.2
4	5	133.5	0.0	0.0	17.1	12.8	+12.8	-------
4	6	168.5	0.0	0.0	50.7	30.1	+30.1	-------
4	7	31.8	0.0	0.0	7.4	23.2	+23.2	-------
4	8	39.0	39.0	100.0	28.8	73.8	−26.2	−25.6
5	0	569.5	369.5	64.9	306.6	53.8	−11.1	−17.0
5	1	11.6	0.0	0.0	0.1	0.0	0.0	-------
5	2	-------	-------	-------	-------	-------	-------	-------
5	3	221.3	77.8	35.2	67.5	30.5	−4.7	−12.8
5	4	319.8	33.6	10.5	148.0	46.3	+35.8	+339.3
5	5	458.4	71.8	15.7	2.9	0.6	−15.1	−96.1
5	6	315.5	11.6	3.7	20.7	6.6	+2.9	+77.6
5	7	50.7	0.0	0.0	25.6	50.5	+50.5	-------
5	8	33.6	0.0	0.0	26.4	78.5	+78.5	-------
6	0	712.4	339.8	47.6	355.6	49.9	+2.3	+4.7
6	1	39.8	39.8	100.0	3.6	9.0	−91.0	−90.5
6	2	57.4	11.4	19.9	11.1	19.3	−0.6	0.0
6	3	300.4	158.0	52.6	184.4	61.4	+8.8	+16.5
6	4	367.3	11.7	3.2	110.2	30.0	+26.8	+837.6
6	5	487.4	86.2	17.7	142.0	29.1	+11.4	+65.0
6	6	1,111.6	171.1	15.4	147.9	13.3	−2.1	−13.5
6	7	134.2	0.0	0.0	34.5	25.7	+25.7	-------
6	8	22.0	11.0	50.0	15.7	71.4	+21.4	+45.5
7	0	368.8	105.5	28.6	140.8	38.2	+9.6	+33.0
7	1	21.1	0.0	0.0	6.8	32.2	+32.2	-------
7	2	34.4	10.9	31.7	1.4	4.1	−27.6	−87.2
7	3	86.3	21.9	25.4	11.2	13.0	−12.4	−48.9
7	4	74.6	12.3	16.5	0.0	0.0	−16.5	−100.0
7	5	10.7	0.0	0.0	0.0	0.0	0.0	0.0
7	6	43.1	21.2	49.2	17.5	40.6	−8.6	−17.5
7	7	2,015.5	85.2	4.2	115.9	5.8	+1.6	+36.4
7	8	229.0	0.0	0.0	12.1	5.3	+5.3	-------
8	0	357.8	65.4	18.3	48.7	13.6	−4.7	−26.0
8	1	57.3	0.0	0.0	0.0	0.0	0.0	0.0
8	2	108.3	31.5	29.1	0.0	0.0	−29.1	−100.0
8	3	45.9	23.6	51.4	0.5	1.1	−50.3	−100.0
8	4	11.6	0.0	0.0	0.0	0.0	0.0	0.0
8	5	11.3	0.0	0.0	0.0	0.0	0.0	0.0
8	6	33.2	0.0	0.0	0.0	0.0	0.0	0.0
8	7	433.3	0.0	0.0	1.8	0.4	+0.4	-------
8	8	1,696.3	100.1	5.9	12.9	0.8	−5.1	−86.9

Results

Table 10 details the results of the 13 sensitivity runs. Results from each run are not discussed individually, but the runs have been related to the four main modal split variables. Trip purpose was not considered as only work trips were evaluated.

Economic Status

The economic status variable indexes the set of 16 out of 80 diversion curves for each purpose that will determine the modal split. The five levels of economic status were determined by the following groupings of median income of workers.

Economic status	*Income ($)*
1	0–2, 499
2	2, 500–3, 999
3	4, 000–5, 499
4	5, 500–6, 999
5	7, 000+

Tests *B* and *M* were designed to analyze the sensitivity of the modal split procedure to variations in economic status. The results show that the modal split procedure is relatively insensitive to changes in economic status. The number of transit passengers declined 2.4 and 4.5 percent respectively in relation to increases of 20.0 and 50.0 percent in median income.

Figure 4 illustrates five modal split curves (percent transit usage vs. traveltime ratio) for the five levels of economic status when the other variables were not changed—level of service ratio 1.25, cost ratio 2.50. For time ratios favorable to transit—shown to the left of the vertical dashed line (traveltime ratio of 1.00) in figure 4—economic status five exhibits a higher split to transit than economic status groups two, three, or four. As time ratios became less favorable for transit, the split to transit became inversely related to income level. In other words, the relationships developed indicate that people having higher incomes are more apt to use good transit than those having low incomes; conversely, people having high incomes are less apt to use poor transit than those having low incomes.

The characteristics noted of these particular modal split relationships tend to explain the relatively small areawide change in modal split in relation to large changes in median income. The modal split procedure indicates that for the zonal interchanges having time ratios favorable to transit, an increase in the split to transit usage occurs according to increases in economic status, as more zones fall into the economic status group five. Conversely, for those interchanges having less favorable time ratios, a decrease in transit usage occurs as median incomes increase. This finding was confirmed by the analysis showing the higher decline in non-CBD oriented trips, 15.4 percent, as opposed to the trips that were CBD oriented, 1.7 percent. These joint effects tended to cancel areawide variation in modal split caused by changes in median income. The modal split procedure appeared to be sensitive to changes in economic status on a zonal basis, but such sensitivity was not apparent on an areawide basis.

Cost Ratio

The cost ratio variable was the ratio of the out-of-pocket costs via each mode. Sensitivity tests *A*, *E*, and *H* were designed to evaluate the sensitivity of the modal split technique to the cost ratio. Results of tests *A* and *H* showed that the range of sensitivity of the cost ratio is fairly narrow; this narrow range was caused by the mechanics of the cost ratio calculation. Four levels of cost ratio are specified by the modal split procedure. Between the first and the fourth levels, the cost ratios were in effect continuous because of interpolation between levels. However, cost ratios lower than those of the lowest level (0.25) or higher than the highest level (3.00) were considered to be equal to the high and low ratios. In other words, a cost ratio of 6.00 or 12.00 was considered to be no different than a cost ratio of 3.00.

Increases in transit fares had only a minor effect on the modal split. When a 15-cent increase in fares was applied to each zonal interchange—from an average base of 35 cents—transit patronage dropped 5.0 percent overall. An examination of the cost ratios from this test indicated that the resultant ratios were predominantly on the maximum level. To prove this, fares were doubled for test *H*, and the decline in patronage from the base was 7.8 percent, or only 2.8 percentage points more than the decline caused by the 15-cent increase. This decline of 7.8 percent probably closely approaches the maximum decline in patronage caused by fare increases that the model would predict. When transit costs were held constant and parking costs in the downtown area were doubled (test *E*), estimates for transit trips to downtown increased 7.7 percent. The modal split procedure was sensitive to changes in the cost ratio only in a very limited range. From the drastic changes in the cost ratio variable—doubled transit fares to double parking costs—the number of estimated riders ranged only from 99,752 to 115,972.

Service Ratio

The service ratio, also termed the excess time ratio, relates the service provided by transit to the service available by private automobile in terms of the portion of the trip time spent outside the means of conveyance (automobile or transit). Tests *C*, *D*, and *I* were designed to evaluate the sensitivity of the modal split technique to changes in the level of service ratio. By factoring the waiting and transfer times for transit by 1.5 (test *D*), transit service was in effect reduced. For example, trains running on 10-minute headways would have 15-minute headways. This change caused a decline in the estimate for transit trips of 15.1 percent. To show the possible variation in estimates of this type, a

Table 7.—Root-mean-square-error analysis of district to district transit work trips

Volume group	Frequency	Total[1]		Mean		RMS error	Percent RMS error
		O–D trips	BPR test estimates	O–D trips	BPR test estimates		
5.0–9.9	13	125.2	99.4	9.6	7.6	13.7	142.4
10.0–14.9	203	2,237.6	1,229.6	11.0	6.1	12.9	117.2
15.0–19.9	3	55.7	103.7	8.6	34.6	24.4	131.3
20.0–24.9	72	1,589.3	1,127.3	22.1	15.7	22.9	103.9
25.0–29.9	10	277.2	382.8	27.7	38.3	43.3	156.3
30.0–39.9	274	9,711.9	8,353.0	35.4	30.5	31.5	88.8
40.0–49.9	112	4,746.6	3,389.2	42.4	30.3	29.6	69.9
50.0–59.9	33	1,822.4	1,186.3	55.2	36.0	34.0	61.6
60.0–74.9	98	6,624.1	6,345.5	67.6	64.8	54.2	80.2
75.0–99.9	90	7,366.9	5,454.5	81.8	60.6	47.5	58.0
100.0–124.9	79	8,961.4	7,914.1	113.4	100.2	62.8	55.4
125.0–149.9	31	4,319.7	3,130.6	139.4	101.0	64.2	46.1
150.0–499.9	187	48,308.0	44,268.4	258.3	236.7	102.0	39.5
500.0 and more	43	34,910.4	31,973.1	811.9	743.6	239.2	29.5
TOTAL	1,248	131,056.4	114,957.5	----	----	----	----

[1] Difference in totals caused by certain movements having zero O–D trips and some model trips.

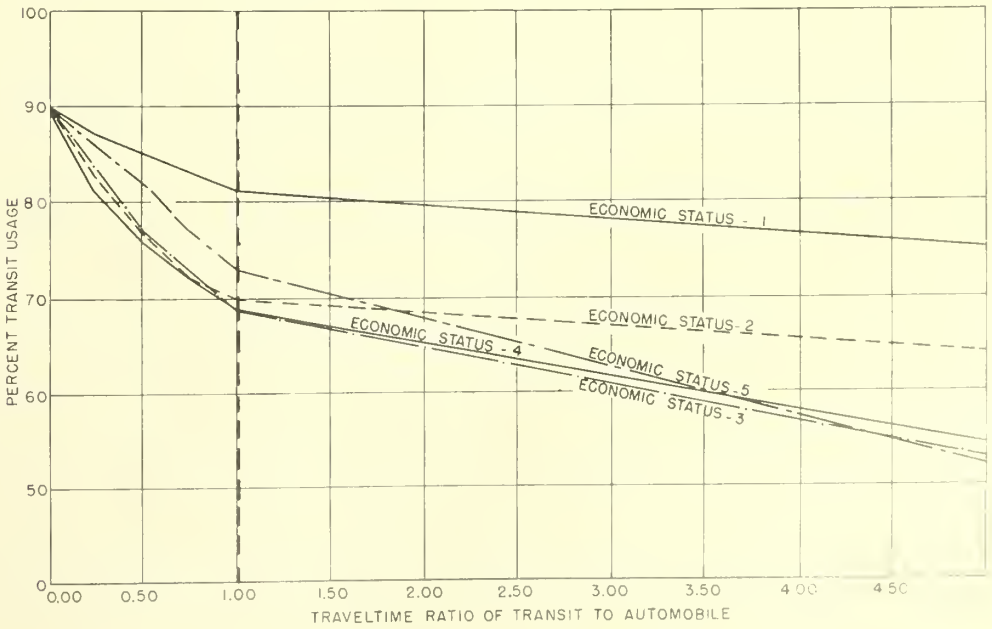

Figure 4.—Variation in modal split related to economic status.

Table 8.—Root-mean-square-error analysis of district-to-district transit nonwork trips

Volume group	Frequency	Total[1]		Mean		RMS error	Percent RMS error
		O–D trips	BPR test estimates	O–D trips	BPR test estimates		
5.0–9.9	4	38.2	34.1	9.6	8.5	11.7	122.3
10.0–14.9	60	660.7	280.7	11.0	4.7	11.0	99.4
15.0–19.9	1	18.2	0	18.2	0	18.2	100.0
20.0–24.9	11	239.7	123.7	21.8	11.2	13.1	59.9
25.0–29.9	3	89.1	58.1	29.7	19.4	18.4	62.1
30.0–39.9	58	2,127.3	1,328.7	36.7	22.9	22.1	60.3
40.0–49.9	18	741.6	609.6	41.2	33.9	23.5	57.0
50.0–59.9	1	56.6	65.4	56.6	65.4	8.8	15.5
60.0–74.9	4	277.1	158.9	69.3	39.7	31.8	46.0
75.0–99.9	7	565.2	347.1	80.7	49.6	36.2	44.8
100.0–124.9	4	452.8	566.9	113.2	141.7	38.7	34.2
125.0–149.9	0						
150.0–499.9	2	347.9	294.9	174.0	147.4	37.8	21.7
500.0 and above	0						
TOTAL	173	5,614.4	3,868.1				

[1] Difference in totals caused by certain movements having zero O–D trips and some model trips.

Table 9.—Cumulative trip length frequency data

Time increment	Accumulated percent of transit trips		Percentage point difference
	1955 O–D data	BPR test estimates	
Minute	*Percent*	*Percent*	*Percent*
0–9	10.85	10.57	−0.28
0–19	41.25	37.90	−3.35
0–29	71.27	69.00	−2.27
0–39	90.93	90.01	−0.92
0–49	97.27	97.60	+0.33
0–59	99.34	99.50	+0.16
0–69	99.89	99.90	+0.01

comparison was made of selected district transit waiting time estimates developed for the 1980 application with estimates developed for the tests applying 1955 data and is shown in table 11. Each of the estimates was made by the same person but at different times.

It is emphasized, however, that the estimates for the BPR test against 1955 O–D data were adjusted to the curve development estimates prior to their use for the test, as previously explained in the material on input factors. To test the sensitivity of the denominator of the service ratio, 1 minute was added to both the automobile parking delay time and the walking time from parking place to ultimate destination (test *C*). This change had drastic effects; transit patronage estimates were increased by 32.7 percent. CBD oriented trip estimates were increased by 10.3 percent. As the waiting and walking times for automobile trips to non-CBD areas were assumed to be 1 minute each for all other tests, only CBD oriented trips are discussed here.

Test *C* indicated such a high sensitivity of the procedure to automobile associated waiting and walking time that an additional test was designed. The parameters developed for these times in the test of the 1955 O–D survey data were substituted for the 1980 estimated times in test *I*. When these 1955 test parameters were used, the estimation for transit trips to the CBD declined 20.8 percent.

The results of these tests indicate that the modal split procedure is highly sensitive to the service ratio within the range of realistic input variables. Because of the relative difficulty in accurately estimating waiting and walking times, the procedure may be too sensitive to this parameter. A determination must be made as to whether walking and waiting times are as pivotal a factor in the choice of a mode of travel as indicated by the sensitivity test results.

As for the cost ratio, fixed limits for upper and lower levels were established for the service ratio. However, unlike the cost ratio, the range of percent transit usage was extremely broad between limits. For example, when the other parameters were held constant at given levels, the modal split might vary from 70 to 50 percent in relation to the maximum change in cost ratio. By varying the level of service ratio between its maximum and minimum values and holding all other parameters constant, a typical range in modal split might be from 70 to 30 percent.

Traveltime Ratio

The traveltime ratio consists of the portal-to-portal time via each mode, including waiting, walking, and transfer times. Sensitivity tests *F*, *G*, and *J* were designed to evaluate the sensitivity of the procedure to traveltime ratio. A review of all three outputs showed that the model was adequately sensitive to changes in traveltime. For trips in the less favorable time ratios, that is, those characterized by the non-CBD oriented trips, the procedure was more sensitive to the traveltime ratio than in the areas having ratios favorable to transit. The maximum areawide change of 13.8 percent in patronage estimates occurred when transit times were factored by 1.5. These output figures showed that the procedure was sensitive to traveltime ratios but that the effects of minor time changes, such as varying speeds on a given route section, would be very hard to detect.

Combined Variables

Sensitivity tests *K* and *L* were designed to determine the joint effect of varying several parameters at the same time. Changing an individual parameter provided a good picture of the relative sensitivity of the procedure to the change in the variable. However, because the modal split procedure exhibited different degrees of sensitivity over certain ranges, it was very difficult to evaluate the joint effect of changes in more than one parameter.

Table 10.—Modal split sensitivity analysis

	Base[1]	A[2]	B[3]	C[4]	D[5]	E[6]	F[7]	G[8]	H[9]	I[10]	J[11]	K[12]	L[13]	M[14]
Total person trips	465,825	465,825	465,825	465,825	465,825	465,825	465,825	465,825	465,825	465,825	465,825	465,825	465,825	465,825
Number via transit	108,169	102,731	103,265	143,586	91,864	115,972	117,287	93,848	99,752	90,308	93,249	76,133	80,571	10,552
Percent diversion	0.2322	0.2205	0.2217	0.3082	0.1972	0.2490	0.2517	0.2015	0.2141	0.1939	0.2002	0.1634	0.1730	0.2265
Percent change		−5.0	−4.5	+32.7	−15.1	+7.2	+8.4	−13.2	−7.8	−16.5	−13.8	−29.6	−25.5	−2.4
Person trips to CBD	148,390	148,390	148,390	148,390	148,390	148,390	148,390	148,390	148,390	148,390	148,390	148,390	148,390	148,390
Number via transit	85,952	82,185	84,463	94,841	76,205	92,609	88,931	81,302	80,078	68,091	80,447	53,915	68,025	84,633
Percent diversion	0.5794	0.5538	0.5692	0.6391	0.5135	0.6241	0.5993	0.5479	0.5396	0.4589	0.5421	0.3633	0.4584	0.5703
Percent change		−4.4	−1.7	+10.3	−11.3	+7.7	+3.5	−5.4	−6.8	−20.8	−6.4	−37.3	−20.9	−1.5
Non-CBD oriented person trips	317,435	317,435	317,435	317,435	317,435	317,435	317,435	317,435	317,435	317,435	317,435	317,435	317,435	317,435
Number via transit	22,217	20,546	18,802	48,744	15,659		28,356	12,546	19,674	22,217	12,802	22,217	12,546	20,889
Percent diversion	0.0700	0.0647	0.0592	0.1536	0.0493	0.0700	0.0893	0.0395	0.0620	0.0700	0.0403	0.0700	0.0395	0.0658
Percent change		−7.5	−15.4	+119.4	−29.5		+27.6	−43.5	−11.4		−42.4		−43.5	−6.0
TOTAL REVENUE	$37,337	$51,030	$35,984	$48,987	$31,647	$39,623	$40,781	$32,009	$68,545	$31,076	$31,634	$26,315	$27,401	$36,564

[1] Base, NCTA run 23, a.m. peak traffic hours, work trips only.
[2] $0.15 added to base fares.
[3] 1.5×median incomes.
[4] 2 minutes added to auto parking and walking times.
[5] 1.5×waiting and transfer times.
[6] 2.0×parking costs.
[7] Transit times factored by 0.75.
[8] Highway times factored by 0.75.
[9] Base fares doubled.
[10] 1955 auto parking and walking times used.
[11] Transit times factored by 1.5.
[12] 1955 parking costs and auto waiting and walking times used.
[13] 0.75×highway times, 0.66×parking costs, 0.66×auto walking time.
[14] 1.2×median incomes.

Table 11.—Comparison of alternate estimates of transit waiting times

District	Transit waiting time estimates from—	
	Curve development	Test made with 1955 O–D data
1	1.2	3.9
6	2.7	4.0
11	3.0	4.7
16	4.6	4.2
22	4.0	2.0
27	7.2	5.6
32	1.7	4.1
37	7.8	8.5

For example, a 25 percent decrease in transit times might cause an 8 percent increase in the estimate for transit patronage if the average cost ratio were 1.5. The same decrease in transit time might have an entirely different effect if, because of changes in fare structure, the average cost ratio were 2.0.

When parameters used in the test of 1955 data for parking costs and automobile waiting and walking times were substituted (test *K*), the estimated transit usage of CBD oriented trips dropped to 36.3 percent from the 1980 base estimate of 57.9 percent. It is difficult to draw conclusions from this particular test as the estimate for CBD oriented transit usage dropped below the level reported in the 1955 O–D survey. In other words, despite the assumptions regarding improved transit, when the 1955 terminal parameters for automobiles were used, the 1980 estimated percent of transit usage was less than the actual 1955 level.

Test *L*, which contained more favorable assumptions regarding 1980 automobile travel conditions—higher automobile speeds, lower parking costs, and shorter walking times—showed a 25.5 percent areawide drop in estimated transit trips. Approximately one-half of this change can be related to the factoring of highway times as test *G*, in which the same highway time change was isolated, showed a patronage decline of 13.2 percent. Because the procedure is much more sensitive to the level of service ratio than the cost ratio, the bulk of the remaining decline can be related to the more favorable assumptions made regarding the walking time from the automobile at the destination.

The results of the analyses for combined variables showed the same key trends as the analyses of individual variables. The trends shown are: (1) The level of service ratio far outweighs the other variables in importance, (2) cost ratio plays a minor role, and (3) travel-time ratio exhibits adequate sensitivity over its entire range.

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(*5*) *IBM 704 Traffic Assignment Program*, by Arthur B. Sosslau (processed), available through the Urban Planning Division, Bureau of Public Roads.

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Comments on Evaluation of New Modal Split Procedure

Comments of three transportation planning officials on the BPR test evaluation made of the new modal split procedure were also presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964. The comments are presented here to provide additional information on the problems being encountered by highway and urban transportation planners in anticipating future needs.

The following material is the comments made by: Thomas B. Deen, Acting Director of the Office of Planning, National Capital Transportation Agency; William L. Mertz, Technical Director, Tri-State Transportation Committee; and George B. Wickstrom, Deputy Director, Penn-Jersey Transportation Study.

Mr. Deen's Comments

A procedure for estimating the relative usage of private and public transportation systems, which is both rational and practical, has long been a pressing need in the urban transportation planning process. The modal split procedure developed for NCTA appears to be a significant step forward in filling this theoretical and methodological gap. The Sosslau-Heanue-Balek article makes a valuable and necessary contribution to a fuller understanding of this procedure, to the implications involved in its application to specific planning problems, and to the subtle interrelationships of the variables affecting transit usage. The authors have treated this complicated and—unfortunately—controversial subject with objectivity and fairness.

Of fundamental importance as a test of its basic validity is the fact that the modal split procedure accurately reproduced the transit usage actually observed in 1955 as regards total areawide transit use, CBD transit use, interdistrict transit use, and transit trip length. It is axiomatic that public transit's greatest strength is in the delivery of work trips to and from the CBD. Estimating transit's ability then to attract CBD-bound workers is essential in proper planning of urban transit and highways systems. The modal split procedure estimated these trips as 75,678, missing the observed O–D survey by only 46 trips, an error so small that it must be considered at least partially coincidence. Total CBD trips, estimated by the procedure were in error by only one percent.

The authors properly point out that non-CBD trips were not so precisely estimated, and follow with the suggestion of development of separate non-CBD modal split relationships and separate handling of such trips. As the model in its present state is a costly and time-consuming procedure, to complicate it further by additional stratification of the thinly-sampled data and to raise the number of modal split curves above the present 160, might not be the most promising approach. Particularly so, as factors other than the need for separate non-CBD modal split relationships may well be more important causes of the lesser accuracy of the non-CBD estimate. These would include the inadequate representation of zonal parameters, such as walking distances and waiting times to employment areas in the larger nonsector zero zones. For example, walking distances to bus stops for each zone—CBD or non-CBD—were estimated so as to represent average conditions to and from trip generation points within the zone. For most nonsector zero zones, such points are primarily residential. However, walking distances to employment or commercial areas within these zones might be quite different from those representing the residential trip generation points.

One disappointing aspect of the model's performance concerns the geographical bias observed in the synthesis of 1955 transit travel. The consistent underestimate of transit usage from the western side of the city and the overestimate on the eastern side are problems of real concern. In order to see the problem in perspective, however, two elements should be considered:

The gravity model trip distribution process used in Washington has been observed to produce a similar bias.[1] Work trips to the

[1] *Evaluation of Gravity Model Trip Distribution Procedures*, by Walter G. Hansen, Trip Characteristics and Traffic Assignment, Highway Research Board Bulletin No. 347, 1962, pp. 67–76.

CBD from the western side of the city were consistently underestimated by the gravity model until adjustment factors were applied. This behavior by the gravity model has been considered a result of unequal distribution of income or other socio-economic factors between the eastern and western sides of the city. No such simple explanation suggests itself in the case of the modal split model as income is one of the input parameters.

The largest percentage of error for any corridor (as indicated by comparison of the 1955 actual and computed figures) was 22.6 percent. However, the largest absolute error was 1,473 trips over a 2.17 hour peak period. If 60 percent of these trips are assumed to occur during the peak hour and the observed peak hour downtown Washington car occupancy (1.8) is used, the error becomes

$$1{,}473 \times 0.60 \times \frac{1}{1.8} = 490$$

vehicles per hour, less than one-third of a highway lane. Considering the limitations in our abilities to estimate future land use, and trip distribution, this would appear to be well within limits acceptable for transportation planning.

The sensitivity tests are extremely interesting and if studied sufficiently can shed much light on the relative importance of the numerous factors affecting transit usage. I concur with the authors' findings that the model is sensitive to cost ratio only through a very limited range, and to service ratios and travel-time ratio through a much larger range. The limited cost-ratio range is not an inherent characteristic of the procedure, however. If data can be found for a broader cost range, then the observed results may be incorporated into this or a similar process.

One surprising element reported from the sensitivity tests is the high elasticity of transit use relative to auto terminal conditions—specifically to auto walk and delay times. I should like to make several comments in this regard:

- One of the most attractive features of unrestricted auto travel that is almost impossible to duplicate with public transportation is that it begins where you want to begin and takes you directly to where you want to go. If, due to lack of properly located parking space, the auto trip must end some distance from the trip destination, then much of the auto convenience is lost, and public transportation is at once more competitive. Ergo, it is not unreasonable to suppose that auto walking time is in fact an important factor in modal split.
- An equally attractive feature of auto travel is that it goes *when* you want to go. If significant delays are associated with unparking—from a parking garage for instance—then auto convenience is reduced. Therefore parking delay could reasonably be supposed to be an important modal split determinant.
- Even if the preceding two points are accepted, the degree of sensitivity shown by the model to auto-walk-wait times would appear to justify careful scrutiny. Perhaps the problem lies in the use of a ratio to express the relative convenience of auto and transit travel. One of the characteristics of the service ratio is that the denominator is significantly smaller than the numerator. In fact, for the nonsector zero-destined trips, the denominator was 2 minutes, while the numerator was usually 7 to 10 minutes or more. Thus a 2-minute increase represents a 100 percent increase in the denominator and a 50 percent decrease in service ratio. A 2-minute decrease makes the service ratio infinite. The extreme effect of this on the modal split can be seen in test *C* where nonsector zero transit trips increased 120 percent as a result of adding 2 minutes to auto-walk-wait times. The same distorting influence is operating for sector zero trips, though to a lesser degree, because the denominator for such trips is larger. Perhaps the problem could be solved by quantifying the convenience factors into time difference—transit excess time minus auto excess time—instead of a time ratio.

Some comment should be made concerning test *K* in which use of 1955 auto terminal values (parking costs, auto-walk-wait times) along with the other 1980 assumptions, including the proposed rail transit system, produced a modal split below the 1955 level. I concur with the authors that this test is "difficult to interpret." However, before any interpretation can be made, certain other items must be fully understood:

- While in this test the *proportion* of peak-hour commuters using transit to sector zero dropped below the 1955 level, the absolute volume of sector zero transit riders held about the same as in 1955.
- The 1955 auto terminal conditions have long since disappeared. Average parking costs have gone up an estimated 100 percent since 1955 due in part to a 30 percent increase in commercial rates, but more importantly to an increase in the number of parkers using pay facilities and a corresponding decrease in the number parking free.
- Test *K*, in addition to assuming an improved transit service, also assumed a substantially improved highway system over 1955, with significant increases in auto travel speed. An intelligent appraisal of the real meaning of returning to 1955 auto terminal conditions in 1980 cannot be made without evaluation of the effects on auto speeds of the shift of such a large number of transit riders to the highway.
- In fact, it can be fairly stated that the modal split at any moment is the result of a large number of conflicting factors that are in equilibrium. Change in any factor, say, parking costs, causes a shift to auto travel. This in turn causes decreases in auto speed, which tends to shift travelers back to transit. Thus the elasticity of modal split indicated in test *K* along with all the other tests must be viewed as somewhat artificial since the feedback required to reach equilibrium has not been accounted for.

In conclusion it must be noted that while the modal split procedure appears promising, there are many elements concerning its use that are as yet unknown. The sensitivity tests reported on here indicate that within limitations the model responds in a reasonable way to changes in input conditions; the 1955 transit use synthesis indicates that the mechanics of the procedure, the method of inputting certain of the variables, and the otherwise questionable procedure of representing conditions within fairly large geographical areas by means of averages (for example, average walking distances, income) are adequate. But results of these tests say little about the relative importance of other factors that probably affect transit use, such as schedule adherence, air conditioning, riding comfort, vehicle esthetics, diesel fumes, subway claustrophobia, station shelters and parking facilities, kiss-and-ride, etc. Nor do we yet know much of the universality of the modal split relationships. Most important, we don't know if modal split relationships remain stable over a period of time. Finally, the entire approach ignores the effect of relative transit and auto use on trip distribution and generation, though logic would seem to say that all these elements are interrelated, at least through land-use changes, and probably more directly as well. All things considered, there are many unknowns worthy of concern and further study. Yet, when viewed alongside the other unknowns in the field of urban planning—the modal split model is a significant advance that contributes much to our understanding of the determinants of public transit usage

Mr. Mertz's Comments

The modal split procedure evaluated by Sosslau, Heanue, and Balek, has received widespread attention in the technical field. Also, the controversy over the future course of highway and transit development in the National Capital Region has assumed national proportions. I am concerned that the technician who reads this paper without more background and perspective might assume a more pessimistic view of the usefulness of the procedure than is warranted. I would, therefore, suggest that the other two papers, *Development of a Model for Forecasting Travel Mode Choice in Urban Areas*, by Von Cube and Hill, and *Application of a Modal Split Model to Travel Estimates for the Washington Area*, by Deen, Irwin, and Mertz, be studied in conjunction with this one.

The problem of mode choice is assuming greater proportions each year. Heretofore, the tools to deal with the problem have been skimpy indeed. The use of time ratio curves alone will no longer suffice. I suggest that the reader make a judgment as to whether the significant factors in modal choice have been incorporated into the procedure. By and large, I submit that they have. We need to know more about the effect of crowding—the standee problem—and other factors in quantifiable terms so more research is certainly needed.

If the reader accepts the position that the major significant variables have been incorporated, the next question to be answered is whether a proper description of the action

and interaction of the variables has been found, which is the subject under discussion. A review of the article and an inspection of the curves will reveal the different sources of data and the portions of the curves for which there were no data at all—the dotted portions. There is always great difficulty in developing a common denominator base both in time and geography for data collected from different sources. This points to the need for the collection of more information in O–D surveys bearing on the mode choice problem. Also, I believe that changes of variables assumed to influence mode choice need to be made within functioning transit-highway systems and results carefully measured and evaluated. The Demonstration Grant Program could be the instrument for such studies.

The reader must also judge the conclusiveness of the test against the 1955 O–D data. The authors state that on a district-to-district basis "the variation between the O–D and model estimates is less than the variation in the survey." At the same time it is stated that the variation between corridor movements is "a particularly critical weakness of the modal split technique." There have been several tests of transportation planning techniques against O–D data in the past. In all cases, the variation has been higher than we would like and leaves moot the question of how much variation is due to sampling and how much is due to a procedure's inability to describe an historical situation.

The sensitivity tests of the model are particularly interesting. It should be borne in mind that the sensitivity tests, by their very nature, extend the parameters into the dotted portions of the curves for which no data were available. The configuration and interaction between variables in these extended ranges, of necessity, were based on logic and intuition. Even so, I am not surprised that changes in fare proved to be less sensitive than, for example, automobile terminal conditions such as parking delay, walking time, and parking costs. I am encouraged that the authors stated that the model is "adequately sensitive to changes in travel time," which has been the backbone of mode selection procedures in the past.

I would like to emphasize a point that none of the three articles brings out clearly. One of the major objectives of this development was to create a modal split computer program to fit into and be compatible with the "so-called" BPR battery of transportation planning programs. This was achieved. The modal split relationships (represented by tables) are input to the program just as travel time matrices, etc. are input. All of the discussion in the article concerns the evaluation of this input. Different sets of relationships were used for the work and nonwork purposes. The program is operational, is compatible, and is usable by any study. The tabular curve relationships should certainly be evaluated against data for the urban area in question and modified or completely reconstructed in the light of local conditions. This operational capability should not be overlooked.

Mr. Wickstrom's Comments

When one is given the opportunity to comment on a material that is in itself a comment on a previous paper, it is difficult to know where to begin. Comments could be directed to the problem (modal split), the method evaluated (diversion), or the evaluating of the modal test methods and results.

In reviewing the conclusions as stated in the paper, major points included: (1) CBD and non-CBD trips may require stratification, (2) certain input variables (notably excess time) are overly sensitive, and (3) further study is required.

Although it is difficult to disagree with these conclusions, I cannot help but feel that the basic approach taken to solve the problem of predicting modal split should also be examined. The approach investigated was a diversion approach; that is, it attempts to predict the percentage of travelers who choose transit rather than the auto. While it may be possible to mathematically match observed transit choice behavior by this method with aggregates of O–D data available, how does one accurately predict the future total number of travelers between two O–D zones and using all modes of travel from which to take a percentage? Isn't the ultimate answer desired not just the percentage on transit, for each corridor, but *how many* on transit or auto?

It would also seem that the data available from the O–D survey were overly stratified in an attempt to introduce as many of the factors that influence transit use as possible. A 5- or even 10-percent sample of CBD trips simply does not permit so many stratifications, as sample variability plays havoc. If home-interview data are ill-suited for models of this type, shouldn't we collect data at the CBD end of the trip?

One of the major reasons for collecting home-interview data throughout the metropolitan area is the present need to obtain a universe of household characteristics and trip interchanges.

Although the day has not yet come when secondary source data and models have made these basic requirements obsolete, perhaps better models could be developed if data collection were intensified in several parts of the urban area to provide a statistically reliable sample for model development purposes, while collecting a slightly smaller uniform sample elsewhere.

If only conventional origin-destination data are available, one is really forced to predict transit use on an area basis and usually forced to ignore or generalize the effects of changes in system characteristics. Modal splits are made before distributing trips rather than afterward.

There is also some question as to whether diversion curves can ever adequately predict modal split, since they usually tend to overemphasize system characteristics at the expense of more determining factors—such as whether or not the wife needs the family car! In this regard, I notice that the model tested did not *directly* deal with car ownership. Yet, the importance of this variable in estimating transit use is illustrated by the fact that only 14 percent of all trips and 9 percent of nonwork trips were made by persons in families owning one car in the Philadelphia area in 1960. For trips by two-car families these low percentages were halved, while persons not owning a car made 76 percent of their trips by transit. It would seem that the apparent effect of car ownership is important enough not to be even partially ignored.

In the Philadelphia area, only 25 percent of all transit trips have origin or destination in the CBD. A CBD derived relationship could not be readily used to explain the remaining behavior, since 75 percent of the trips would then be estimated on the basis of relationships derived from 25 percent of the trips.

These comments were not directed at criticising the model, but rather to point up the difficulties inherent in deriving models of this type with conventional origin and destination survey data. The fact that the model behaved as well as it did underlines the need for continued study along these lines. This is the only modal split model now available that has been derived and tested with O–D data from several cities and that deals directly with relative transportation system characteristics. If the excess time factor were modified or eliminated, it would serve as an important interim tool while awaiting the results of the further research recommended.

The paper presented by Mr. Sosslau is an excellent example of the type of thorough, painstaking model evaluation required before the problem of modal split can be solved. The authors have done an excellent job.

Motor Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in the table on pages 10–11. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of December 31, 1963, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit truck, truck-tractor semitrailer combinations, and other combinations.

New Publications

Highway Statistics, 1962

The Bureau of Public Roads, U.S. Department of Commerce, has published a new 180-page bulletin, *Highway Statistics, 1962*, the 18th in the annual series that presents statistical and analytical tables of general interest on motor fuel, motor vehicles, highway-user taxation, State and local highway financing, road and street mileage, and Federal aid for highways. This issue also includes several tables giving the legislative provisions that govern the disposition of State highway-user receipts.

Highway Statistics, 1962, may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for $1.00 a copy. Some of the previous annual issues of the series and the summary to 1955 are also available from the Superintendent of Documents; a list of available issues is carried on the inside back cover of this magazine.

Road-User and Property Taxes on Selected Motor Vehicles, 1964

The publication, *Road-User and Property Taxes on Selected Motor Vehicles, 1964*, is available from the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402, at 45 cents a copy.

This 56-page bulletin has the same objectives as the four previous reports (1950, 1953, 1956, and 1960). Taxes are discussed on each of a group of motor vehicles that had been selected to illustrate and compare the taxes that apply to vehicles of different types in each State. Basic data on road-user taxes and property taxes as of January 1, 1964 are supplied to give both a direct measurement of the impact of the taxes on different vehicles, and place these taxes in proper perspective. No attempt has been made to assess the merits of the taxes imposed nor to indicate support for or opposition to any tax policy.

Calibrating and Testing a Gravity Model With a Small Computer

The Bureau of Public Roads, U.S. Department of Commerce, has published a manual titled *Calibrating and Testing a Gravity Model With a Small Computer*. This publication explains and illustrates the theory and use of a system of analytical procedures and computer programs for calibrating and testing a gravity model trip distribution for a small or medium sized urban area using the IBM 1401 (16K) computer. The IBM 1401 programs are completely compatible for use on the IBM 1410 (40K) computer if the IBM 1410 is equipped with a compatability switch. By combining these programs with those concerned with traffic assignment, it is possible to complete most of the analytical phase of a comprehensive transportation study. The manual references a compatible set of IBM 7090 computer programs for traffic assignment. The 1401 battery does not presently contain traffic assignment programs.

Calibrating and Testing a Gravity Model With a Small Computer, may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for $2.50 a copy.

U.S. GOVERNMENT PRINTING OFFICE:1964

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–32 are available upon request addressed to Bureau of Public Roads, Washington, D.C., 20235.*

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REPORTS TO CONGRESS

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Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:

First Progress Report, House Document No. 106 (1957). 35 cents.

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

Final Report, Part VI: Economic and Social Effects of Highway Improvement, House Document No. 72 (1961). 25 cents.

The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

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Map of U.S. showing routes of National System of Interstate and Defense Highways, Federal-Aid Primary Highway System, and U.S. Numbered Highway System. Scale 1 inch equals 80 miles. 25 cents.

PUBLICATIONS

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

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Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

PUBLICATIONS—Continued

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Highway Statistics, Summary to 1955. $1.00.

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Interstate System Route Log and Finder List. 10 cents.

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Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.

Part V—Traffic Controls for Highway Construction and Maintenance Operations (1963). 25 cents.

Parking Guide for Cities (1956). Out of print.

Peak Rates of Runoff From Small Watersheds (1961). 30 cents.

Road-User and Property Taxes on Selected Motor Vehicles, 1964. 45 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1958: a reference guide outline. 75 cents.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP–61 (1961). $2.25.

Standard Plans for Highway Bridges (1962):

Vol. I—Concrete Superstructures. $1.00.
Vol. II—Structural Steel Superstructures. $1.00.
Vol. III—Timber Bridges. $1.00.
Vol. IV—Typical Continuous Bridges. $1.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Aerial Surveys in Highway Engineering (1960). 40 cents.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Transition Curves for Highways (1940). $1.75.

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IN THIS ISSUE

The Discovery Process in Highway Land Acquisition, *by Sidney Goldstein, James Rice, and William Lavelle* 21

An Evaluation of Partial Taking of Property for Right-of-Way, *by G. V. Broderick and F. I. Thiel* 38

New Publications 44

TITLE SHEET, VOL. 32

The title sheet for vol. 32, Apr. 1962–Feb. 1964, of PUBLIC ROADS magazine is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by a request to the editor of the magazine, Bureau of Public Roads, Washington, D.C., 20235.

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The Discovery Process in Highway Land Acquisition

BY THE
ECONOMIC RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by SIDNEY GOLDSTEIN, Chief, Socio-Economic Research Branch, and JAMES RICE and WILLIAM LAVELLE, Research Assistants

Research on the current trend in court decisions regarding the use of the discovery process in condemnation cases (procedures by which one party may obtain information from an adverse party prior to actual trial) was undertaken by the Bureau of Public Roads to provide immediate information to highway lawyers, appraisers, right-of-way personnel, engineers, and others concerned in land acquisition for construction of the National System of Interstate and Defense Highways and other Federal-aid highways. It was believed that the latest information would be helpful to these highway personnel in establishing the legal boundaries for the discovery of appraisers' names, file materials, facts, and opinions. About 125 condemnation cases, as well as the relevant statutes in all States, covering the past 20 years were reviewed and analyzed as the basis for an evaluation of procedural developments in the use of discovery procedures.

A comparison is presented in this article of Federal and State developments since the Federal Rules of Civil Procedure have been effective (1938), and the influence of the Federal rules on State statutes and procedures is discussed. Detailed information is presented on cases and statutes involving discovery of expert fact and opinion, books, documents, and the attorney's work product. The attitude of the courts toward the government as a party is given particular attention.

This research indicated that a tendency exists toward liberalization of the use of discovery procedures in eminent domain proceedings. Development of the discovery doctrine has a long history, but only recently has this doctrine affected right-of-way litigation to any appreciable extent. As other substantive fields of law have adapted to new procedures, a compatibility of discovery procedures and eminent domain law is expected to develop. The authors point out that in those few States where discovery of opinion as well as fact is permitted, the litigants' best defense against inordinate use of the discovery process is well-prepared documents, papers, and other materials. In most jurisdictions however, opinion of appraisers, work papers, etc. are protected as being part of the attorney's work product or are subject to other judicial or legislative protection against disclosure.

Introduction

AN EVALUATION of the fundamental nature of the process of discovery in litigation and its significance in the land acquisition process is presented in this article. This evaluation is expected to provide helpful information to legal counsel and right-of-way personnel in the Bureau of Public Roads, other Federal agencies, and State highway departments on the development of adequate procedures to: (1) counter the inordinate use of discovery rules against State highway departments and the Federal Government in eminent domain cases, and (2) encourage the use of the discovery procedure where it would be relevant to a determination of fair compensation for the property owner.

Of particular significance to agencies concerned with public works programs is land acquisition and their courtroom efforts. About 10 percent of highway land acquisitions culminate in trial. In the normal process of trial, standard rules of evidence apply, but even these evidentiary rules, accepted over the years, have caused the erection of barriers against a complete exchange of information between litigants.

A new area of legal effort has been created by the acceptance of the Federal Rules of Civil Procedure (*1*)[2] and the discovery rules included therein. Discovery is applicable to every substantive field of civil law. Hence eminent domain proceedings are not exempt from the discovery rules. Therefore, these rules have affected the statutory and the case law of eminent domain. In some respects, discovery meets different problems in this field, for in condemnation sovereignty and public interest are involved. Many questions arise: Should the exchange of information be the same in condemnation as in other types of litigation. Are States and the Federal Government reachable by discovery to a greater extent than private citizens. Does the application of discovery rules to the field of eminent domain, and experts such as appraisers, etc., introduce a new dimension to the problem that makes it necessary for public personnel to be more painstakingly file conscious than they otherwise would need to be.

The nature of discovery rules produces considerable concern of both private and public litigants as to the contribution of the process to the fairness of courtroom proceedings. Public officials, at both the Federal and State levels, have been concerned with the possibility that inadequate data will be disclosed and that the secrecy of trial preparation will be disturbed (*2*).

Background

Legal philosophers and practitioners have held different views with respect to the purpose of legal proceedings. Legal practice has been considered by some to be a substitute for trial by battle, a medieval institution from which a more refined adversary procedure has evolved. To others, adversary practice has represented the only way to settle a dispute; that is, with the most efficient counsellor one who can make the most telling presentation of facts at his command and one who is able to effectively persuade the arbiter, court, or jury.

This attitude toward adversary procedure when related to another of the substantive and philosphical goals of the legal process, to do justice or to pay fair compensation in the field of land acquisition, has caused a gradual reduction in the opposition process. This has been accomplished through attempts at pretrial conferences and discovery procedures, which are judicial processes to reveal pertinent information to opponents before trial, and

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

[2] References indicated by italic numbers in parentheses are listed on page 30.

through other legal mechanics. One objective of these mechanics, of course, is to reduce the courtroom drama.

Land acquisition programs

Public improvement agencies are intimately involved in land acquisition. The Federal-Aid Highway Act of 1956, for example, created a tremendous land acquisition program for both the Federal Government and the individual States. It is anticipated that more than 750,000 individual parcels of land will have been acquired by 1972 for the National System of Interstate and Defense Highways at a cost of almost $7 billion. The ABC system of highways will probably affect twice this number of parcels by the same date (*3*). Other public improvement programs involving urban renewal, housing, reclamation, flood control, etc., also are involved in such proceedings.

Individual government units, at both the local and State levels, and Federal Government units may be engaged in litigation arising from the exercise of their eminent domain powers at any time. Many of these government units are concerned with the applicability of general evidentiary rules to right-of-way acquisition and litigation. Questions regarding discovery of expert witness materials, appraisal records, appraiser names, or evidentiary reports and data have recently arisen in right-of-way litigation because the number of these cases has increased.

Discovery tactics have a long history dating back to the bills of discovery used in equity courts. To ensure fairness in trial, different methods have been built upon the equity experience in Federal and State jurisdictions, involving both pretrial and discovery procedures. Wherever discovery procedures are in use in the United States, they are based upon the early experience with California and New York codes; these codes are the basis for the discovery provisions of the Federal Rules of Civil Procedure. In turn, the reciprocal influence of the Federal rules upon State procedure is apparent today and the tables in appendix III indicate the similarity of these statutory provisions. In general, discovery rules are being interpreted very liberally today, which has consequent implications for public agencies in land acquisition.

To evaluate these discovery implications for eminent domain proceedings, information is presented on such items as: (1) the scope of discovery examination, (2) when and how these examinations may be made, (3) the discoverability of government information, and (4) expert testimony and the lawyers' work product.

The Discovery Process

The nature of discovery

Discovery is a judicial process conducted, prior to an actual trial, according to rules of procedure adopted by the courts or legislature of a particular jurisdiction and overseen by the judge of the court in which the action is pending. The basic purpose of this process is to furnish pertinent information to one adverse party that the other party may have in his possession or control. Modern discovery procedures may be conveniently dated from the inception of the Federal Rules of Civil Procedure in the United States District Courts, which became effective Sept. 16, 1938. The idea of discovery, however, is much older; it originated in the early equity courts as a bill of discovery (*4*), as well as in some of the early code States such as California and New York. The purpose of the bill was to enable the party to prove his own case, not to disprove the case of his adversary (*5*). This same criterion has been set by the courts today.

The Federal rules, for example, provide for discovery and pretrial procedures in rules 16, 26–37, and 45. Appendix I is a compilation of the rules highlighted in this article—rules 16, 26(b), 30(b), 30(d), 31(d), 33, 34, 45(b), 45(d), 71A(a). Appendix III shows that the Federal courts and all 50 States have some provision for discovery procedures, and that a majority of the States has adopted either the Federal rules per se or has very similar statutory provisions for deposition and discovery procedures.

Each of the rules cited is interrelated, and they must be construed together (*6*). This principle is particularly true in those States that have adopted the Federal rules or have substantially the same provisions (*7*). Discovery by written or oral interrogatory, deposition, or production of documents or other tangible things is not limited to the specific provisions of rules 26 to 37, but rather by an understanding of these rules, as well as rules 16, 45(b), and 45(d), as one interrelated process. In many ways pretrial procedures and discovery procedures are synonymous.

Federal rule 16 provides for a pretrial conference to consider: (1) The simplification of issues. (2) The necessity or desirability of amendments to the pleadings. (3) The possibility of obtaining admissions of fact and of documents that will avoid unnecessary proof. (4) The limitation of the number of expert witnesses. (5) The advisability of a preliminary reference of issues to a master for findings to be used as evidence when the trial is to be by jury. (6) Such other matters as may aid in the disposition of the action (*8*).

As shown in appendix III, 37 States and the District of Columbia presently have some provision for pretrial conferences. The majority of these State jurisdictions has followed the provisions of Federal rule 16 verbatim. In California, Connecticut, Iowa, Maryland, North Carolina, and New Jersey, the scope of the pretrial conference has been expanded to specifically include matters not included under the Federal rule. The applicable rule in the 1962 Code of Iowa, shown in appendix II, is a good example of such expansion. New York, however, is among those States that make no provision for a pretrial conference (*8*). Nonetheless, pretrial conference exists in New York as a matter of judicial practice.

Pretrial

The purpose of the pretrial conference and procedure has been described in different ways, but the majority of these explanations appears to arise from the six considerations listed in the text of Federal rule 16. All of the States that have a pretrial provision list similar considerations. The proper application of discovery includes the use of rule 16, as well as the deposition and discovery procedure in rules 26–37, coupled with the subpoena power of rule 45.

As land acquisition involves the use of technical terms and expert witnesses, pretrial procedures tend to reduce the amount of trial preparation and court time required (*10*). Thus, discovery can be used in land acquisition cases (1) to simplify the issues before the actual trial, (2) to arrive early at a fair market value through the use of appraisal reports and taking of depositions of appraisers and other qualified expert witnesses, and (3) to satisfy the constitutional requirement of just compensation. The purpose of pretrial preparation is not to harass the adverse party or merely to uncover the mistakes or weaknesses of the opponent but to balance the interests of both parties so that the proceedings may be expedited (*11*). By pretrial conferences the possibility of settlement is greatly increased, unnecessary expense to the parties may be eliminated, and fair treatment may be given to both the landowner and the condemnor. Thus, the problem of crowded court dockets might be alleviated. Concerning the advantages of pretrial, it has been said that: "Pretrial is now generally considered one of the accepted means of obtaining the fullest possible knowledge of the issues and facts before trial. It and the whole system of discovery help us find the truth, and that is what a lawsuit is intended to do under our system of justice under law" (*12*).

Consideration of the implications of pretrial and discovery procedure, particularly in land acquisition proceedings, has been discussed recently in articles and speeches; some of them are: a paper by Micah H. Naftalin concerning the development of pretrial practice in State condemnation cases, presented at the 40th annual meeting of the Highway Research Board (*13*); J. D. Buscher, special assistant attorney general in Maryland, spoke of the favorable experience of the Maryland State Roads Commission with discovery practice at the 1962 Workshop on Highway Law (*14*); and John P. Holloway, assistant attorney general, Colorado Department of Highways, discussed pretrial discovery at the annual Western Association of State Highway Officials (WASHO) Conference in 1962 (*15*). In addition, the California Law Revision Commission has instituted a recent study concerning Pretrial Conferences and Discovery in Eminent Domain Proceedings.

Although the Federal rules on discovery have been used in land condemnation litigation, albeit with some confusion, since their inception, they were made applicable to land condemnation in 1951 by direct provision in Federal rule 71A, Condemnation of Property, which reads: "(a) Applicability of Other Rules. The Rules of Civil Procedure for the United States District Courts govern the procedure for the condemnation of real and personal

property under the power of eminent domain, except as otherwise provided in this rule." Prior to the adoption of this provision, procedure in Federal courts suffered from a lack of uniformity in the applicability of Federal statutes to condemnation procedures. Rule 71A(a) represents an effort to provide uniformity (*16*). Accordingly, it has been held that discovery is available in condemnation proceedings (*17*). The feeling was recently expressed that all opinions in condemnation cases prior to the adoption of rule 71A(a) were mere dicta (*18*).

Alaska, California, Delaware, Kentucky, Maryland, and Missouri have adopted provisions similar to the Federal rule (*19*). In relating discovery practice to condemnation cases, Illinois passed the Civil Practice Act of 1956, which made the rules of discovery, including sanctions, appropriate in condemnation cases (*20*). Other States, which have recently adopted the Federal rules or a similar version but have not adopted rule 71A, have recognized and reserved it for future legislation. This recognition together with recent court interpretations favoring liberal construction (*21*) of these States' rules, following the spirit of the Federal rules, indicates that the use of discovery in condemnation cases is a new and growing combination of procedural and substantive law.

Scope

The specification of the unity of pretrial and discovery practice and the reference in the Federal rules and elsewhere to the use of this practice in condemnation indicates a public policy in favor of discovery in general. But such a policy has limitations; therefore, this article indicates what is discoverable.

Very often the condemnor, a public agency, is concerned with the confidentiality of its internal materials, which may have usefulness in a particular trial. A State highway department, for example, in preparation for condemnation proceedings, may have gathered appraisal reports prepared by an expert in anticipation of litigating the issue of just compensation. Similarly, the opinion of the appraiser, the factual material gathered as to the value of the land taken, as well as the highest and best use of the remainder parcel, are items that might be best to hold until trial. Also, matter may be in the agency files that is absolutely senseless and incompetent and being poorly drawn should not be exposed to public scrutiny. Furthermore, the condemnee may wish to obtain such information because he believes that it would make the best case for him. In fact, the success or failure of a case may sometimes hinge on the pretrial discovery stage despite the courtroom expertness of the attorneys.

The limits of examination by deposition and discovery for cases pending in the United States District Courts, are set forth in rule 26(b) of the Federal rules. Although this rule reads as though it defines the scope of depositions, it also sets forth the limits for the entire discovery procedure. Rule 26(b) provides that: "Unless otherwise ordered by the court as provided by rule 30 (b) or (d), the deponent may be examined regarding any matter, not privileged, which is relevant to the subject matter involved in the pending action, whether it relates to the claim or defense of the examining party or to the claim or defense of any other party, including the existence, description, nature, custody, condition, and location of any books, documents, or other tangible things and the identity and location of persons having knowledge of relevant facts. It is not ground for objection that the testimony will be inadmissible at the trial if the testimony sought appears reasonably calculated to lead to the discovery of admissible evidence."

As shown in appendix III, several States have adopted rule 26(b) without any substantial additions or deletions. Illinois, South Dakota, and Tennessee have adopted it with the exception of the last sentence. Idaho, Maryland, Minnesota, New Jersey, Washington, and West Virginia have added to their comparable rule 26(b) the proposed amendment to the Federal rules. Also, California and Maryland have made other additions to their version of rule 26(b).

Under rule 26(b), the examination is not limited to a party to the action, but any person having knowledge of relevant, unprivileged facts can be examined. But under rule 33, Interrogatories to Parties, and rule 34, Discovery and Production of Documents and Things for Inspection, Copying, or Photographing, only the adverse party may properly be examined. The scope of examination permitted under rule 26(b) is not extended into rules 33 and 34, and the limits of the examination as to persons are prescribed by each individual rule.

Discovery and Land Acquisition

One of the most troublesome areas of the discovery rules with regard to land acquisition involves the determination of the proper scope of examination as to subject matter. Any matter, not privileged, which is relevant to the subject matter of the pending action is properly discoverable according to rule 26(b). The courts, however, have been far from unanimous in their delineations of the scope of discovery, especially in the interpretation of the meaning of any matter.

Fact and opinion

The most significant point, especially for appraisal records in land acquisition cases, is the distinction between fact and opinion. Although no such distinction is made in the rules, the courts have interpreted rule 26(b) as if there were. The consequences of these holdings are far reaching and will be given detailed consideration later in the article.

Before a court makes a decision as to whether the material sought constitutes fact or opinion, the extent to which one party must divulge the identity and location of persons having knowledge of relevant facts is determined. Discovery of such persons is specifically provided for under rule 26(b) (*22*). The court must decide whether the person taking the deposition is actually seeking the identity of persons having knowledge of relevant facts or whether he is attempting to obtain a list of the witnesses that his opponent intends to call at the trial. Apparently, it is permissible under rule 26(b) to inquire into the identity and location of persons having knowledge of relevant facts for the purpose of discovery (*23*). This provision must not be confused with an attempt to obtain a list of witnesses whom the adverse party intends to call at the trial, however. Discovery has sometimes been denied where the object was to obtain a list of witnesses to be called (*24*).

A New York court has ruled that a showing of some special circumstances will justify nonadherence to the general rule prohibiting discovery of witnesses (*25*). But disclosure of witnesses has been ordered by some courts (*26*). A New Hampshire court, for example, in a personal injury action, ordered disclosure of a list of names and addresses of witnesses; the court ruled that those witnesses were not the exclusive property of either party and that, in the interest of justice, their testimony should be introduced in the action (*27*).

Protection of Expert Witness

Whether the names of witnesses must be disclosed has an important bearing on land acquisition cases, because many of the witnesses to be called will be appraisers and other persons having expert knowledge of the subject matter. Consequently, those persons having knowledge of relevant facts will often coincide with the witnesses to be called at the trial. To compel disclosure in accordance with rule 26(b) would often violate the provision denying discovery of the names of potential witnesses. Once the appraiser's identity had been disclosed, he would be subject to the full range of the discovery procedure as to his knowledge of the property. The courts have generally held that the condemnor could refuse to answer interrogatories seeking the names, addresses, and positions of persons who had aided in compiling the appraisal data (*28*).

New Jersey, recognizing this problem and attempting to protect the expert witness, amended their rule 26(b) in 1955. Their rule 4:16–2, states: "A party may require any other party to disclose the names and addresses of proposed expert witnesses; except as provided in R.R. 4:25–2, such disclosure shall be solely for the purpose of enabling the party to investigate the qualifications of such witnesses in advance of trial."

In 1960 a New Jersey court handed down an interpretation of its rule. In a personal injury action, the plaintiff was required to disclose the name and address of his expert witness, but the defendant could not take the deposition of the expert as to facts within his knowledge on the theory that the expert was a person having knowledge of relevant facts (*29*). The court decided that the language of the rule was clear and interpreted the word "solely" in a literal fashion. By

analogy, this decision indicates a policy of protecting the work of an appraiser from discovery, even though the appraiser's name must be disclosed.

Some discrepancy also exists as to whether information that is known by or is equally available to the interrogator is discoverable. One position is that the interrogator is not limited to facts exclusively or peculiarly within the knowledge of the adverse party, even when the interrogator has at his disposal an adequate or even better source of information (*30*). As recently as 1959, however, in a condemnation proceeding a California court held that, although there was no valid objection to the discovery of relevant, unprivileged factual data, discovery would be denied as the data was readily available to the defendant by other means (*31*). But some States have adopted by statute a more liberal approach and try to remove some of the uncertainty surrounding the proper scope of examination. Thus, New Jersey and Idaho have added the following sentence to their rules governing the scope of examination: "Nor is it ground for objection that the examining party has knowledge of the matters as to which testimony is sought." (*32*)

In compelling discovery of matters already within the knowledge of the interrogator, the rationale sometimes used by the courts is that a party is entitled to elicit such information for the purposes of cross-examination or for the purpose of impeaching the creditability of the witness at the trial. The converse reasoning is that discovery constitutes an invasion of the work product of an expert or attorney or that it would give to one party a free ride and promote laziness. These contentions are discussed more fully in another part of this article. With regard to the discoverability of matter that one party intends to use as evidence in establishing his case, a New Jersey court has held that the moving party cannot be allowed to pry indiscriminately into the opponent's case to ferret out evidence by which the case will be proved (*33*).

Furthermore, matters are discoverable if they are not privileged. The uncertainty surrounding the concept of privilege is significant in land acquisition cases where the expert witness is important. Thus, information gathered by an appraiser in the preparation of a land acquisition proceeding has been held to be privileged matter (*34*) that need not be disclosed either at the time discovery is sought or at the trial; but, this is not a unanimous holding. Attempts have been made to place such reports within the scope of the immunity set forth in the *Hickman* v. *Taylor* case (*35*), although other courts have rejected such an interpretation. This case is invariably the basis upon which the Federal courts base their reasoning, but many State courts rely upon the amendment proposed to the Supreme Court. (See p. 28.) to limit the scope of the examination and to protect a party's expert witness from the necessity of disclosing information. As enacted by Idaho, the amendment to rule 26(b) reads as follows.

"The deponent shall not be required to produce or submit for inspection any writing obtained or prepared by the adverse party, his attorney, surety, indemnitor, or agent in anticipation of litigation and in preparation for trial unless the court otherwise orders on the ground that a denial of production or inspection will result in an injustice or undue hardship; nor shall the deponent be required to produce or submit for inspection any part of a writing which reflects an attorney's mental impressions, conclusions, opinions or legal theories, or, except as provided by rule 35, the conclusions of an expert."

Although the amendment does not make mention of any privilege that is to attach to the writing of any of the enumerated persons, some courts have read it as if a privilege were granted.

A third major area of disagreement as to the scope of examination concerns the sentence of rule 26(b): "It is not ground for objection that the testimony will be inadmissible at the trial if the testimony sought appears reasonably calculated to lead to the discovery of admissible evidence." The Federal courts, in accordance with the policy of liberal construction of the rules, allow a wide range of discovery, and discovery is permitted of what might normally be regarded as inadmissible evidence.

South Dakota, which has adopted a substantial portion of the Federal discovery procedure, but has not included that sentence in rule 26(b), has developed the unique position that the discovery rules are to be interpreted individually as to scope (*36*). Pennsylvania has recently restricted discovery to the bounds of the trial itself regarding evidence (*37*). In New Hampshire, although the Federal rules have not been adopted, the State's supreme court has held that the liberal interpretation given to the rules allows evidence to be discovered, although it may be inadmissible at the trial (*38*).

Relevancy

The development of rule 26(b) through court interpretations has led ultimately to the establishment of relevancy as the basic criterion for determining the scope of a discovery examination. Relevancy is not generally to be equated with relevant as ordinarily used in the admissibility of evidence. Rather, the relevancy of the subject matter is the test, and subject matter is broader than the precise issues presented by the pleadings (*39*). Elsewhere the real test is considered to be whether an answer would serve any substantial purpose, either in leading to evidence or in narrowing the issues (*40*).

With such a vague definition of relevancy, control of the discovery procedure in effect rests with the discretion of the court. For example, the discovery of documents was denied in a recent action under the Federal Tort Claims Act, because a "minimal showing of general relevancy and no more" was not considered a sufficiently good cause for compelling disclosure (*41*). Besides the several restrictions incorporated into rule 26(b), there are provisions in the subsequent rules that vest the courts with the authority to issue protective orders for the benefit of the deponent.

Rule 30(b) sets forth a number of specific orders that the court may issue at its discretion upon a showing of good cause by the person to be examined. It is also provided that the court may make any other order that justice requires to protect the party or witness from annoyance, embarrassment, or oppression. With this broad power, a court can substantially control the scope of the discovery procedure.

Good cause

In keeping with the policy that all of the rules for discovery are to be read *in pari materia*, these protective provisions were specifically incorporated into Federal rules 31, 33, and 34, thereby giving the courts wide discretion in every aspect of the discovery procedure. Rule 30(b), which concerns Depositions Upon Oral Examination, requires a showing of good cause by the deponent before one of the restrictive orders will be issued. The courts have generally interpreted this as implying that depositions may be had as a matter of right, and that they can only be denied for good cause shown. As the rules are to be liberally construed to effect a greater measure of discovery, the courts have shown some reluctance in issuing any orders that would narrow the scope of the examination and inhibit the discovery procedure.

Rule 33, providing for Interrogatories to Parties, permits any party to "serve upon any adverse party written interrogatories to be answered by the party served or, if the party served is a public or private corporation or a partnership or association, by any officer or agent, who shall furnish such information as is available to the party." Both the scope of examination of rule 26(b), and the restrictive provisions of rule 30(b) are applicable to interrogatories. As under rule 30(b), the serving of interrogatories to be answered by an adverse party is considered by the courts to be a matter of right so that a protective order will be granted by the court only upon a showing of good cause by the party interrogated.

In some condemnation cases, objections have been made to certain interrogatories propounded in accordance with rule 33. These objections were overruled because the purpose was no more than to ascertain the existence of documents supplied to the appraisers (*42*). Similarly, discovery was also permitted where the moving party sought a list of the sales of properties that might have been or should have been considered in reaching an evaluation of the property (*43*).

The Work Product

Most litigation involves matters of evidence that are solely within the knowledge of the individual attorneys. This evidence is developed as part of the trial preparation and in pursuit of the confidential relation between attorney and client. Names of witnesses, testimony, and individual statements of fact

are included in the record of an attorney for trial presentation. In this respect, courts have tended to expand the privilege of attorney-client to work papers that are required for the litigation of the case. Questions then arise as to: how far this attorney-client privilege extends; whether all documents and facts within the knowledge of experts or other witnesses should be required; whether some way exists to disengage the attorney's work papers from those of witnesses, and if so, does it violate the attorney-client relationship.

This subject has been discussed fully in the recent literature because of a 1947 case that answered many questions and raised a number of others. The interpretations of additional points since 1947 have provided the rationale for both Federal and State interpretations of these issues.

Landmark case

The landmark case, *Hickman* v. *Taylor*, arose as the result of an accident involving the sinking of a tugboat. After the claim had arisen but before the action was instituted, the plaintiff's attorney filed many interrogatories on the defendant under rule 33. One interrogatory inquired whether any oral statements of members of the crew were taken in connection with the accident and requested that exact copies of all such statements be attached, and that the defendant set forth in detail the exact provisions of any such oral statements or reports. The defendant refused and was held in criminal contempt by the District Court. The court permitted discovery (*44*) on the rationale that discovery of all matters relevant to a suit should be allowed to the fullest extent consistent with orderly and efficient functioning of the judicial process. Also, the court held that the mere fact that statements of third parties have been taken by the attorney does not of itself give rise to the traditional privilege accorded to communications between attorney and client.

The Court of Appeals reversed the District Court and coined the concept of the *work product of the attorney* (*45*). This concept represented a new extension of the traditional privilege afforded to the attorney-client relationship by United States courts, though it was already firmly rooted in English law (*46*). The Supreme Court (*47*) rejected the extended privilege theory but accepted the new category of work product on a public policy basis and denied discovery of the material sought. As a result, the continuing problem of the scope of the work product was initiated. The Supreme Court in *Hickman* v. *Taylor* spoke of this problem as "a problem that rests on what has been one of the most hazy frontiers of the discovery process"(*48*).

The Court qualified the work product category, and thus distinguished it from the absolute category of privilege with the following explanation: "We do not mean to say that all written materials obtained or prepared by an adversary's counsel with an eye toward litigation are necessarily free from discovery in all cases. Where relevant and non-privileged facts remain hidden in an attorney's file and where production of those facts is essential to the preparation of one's case, discovery may properly be had. . . .

"Were production of written statements and documents to be precluded under such circumstances, the liberal ideals of the deposition-discovery portions of the Federal Rules of Civil Procedure would be stripped of much of their meaning."(*49*)

In other words, some material may be classified as the attorney's work product and still be discoverable; whereas, any material that is privileged is per se nondiscoverable.

Necessity

The conditions by which work product material may be discovered are based primarily upon a showing of necessity (*50*). Necessity, however, may be shown although it cannot be specifically defined so as to provide a formula to determine when discovery will be permitted or denied. "Good cause is equivalent to necessity. It usually consists of a combination of need factors which justify discovery of what would otherwise not be discoverable. There is no all-embracing practical formula and definitions are of relatively little help." (*51*)

Need factors have been predicated upon such considerations as the demands of justice, the purpose for which the material is sought, whether it is essential to the litigation, whether it is otherwise available, and whether undue hardship would result if discovery were denied. Thus the question of necessity becomes circuitous and rests ultimately upon the discretion of the court for a determination.

The scope of the holding in the *Hickman* v. *Taylor* case is explicitly limited to only the trial preparations of attorneys and does not include the work product of experts such as land appraisers, economists, realtors, and experts in general. The 1946 proposed amendment to the Federal rules was designed to provide for reports of experts. The U.S. Supreme Court, however, rejected the Advisory Committee's proposal and handed down the decision for omitting experts and parties other than the attorney. In 1949, the vacuum was filled by an extension of the Hickman rationale in *Alltmont* v. *United States* (*52*), an action against the United States Maritime Commission for personal injuries. The Court of Appeals reversed the District Court and held that it was improper to construe Admiralty rule 31, which is the same as Federal rule 33, as permitting the libellants, without any showing of good cause, to compel the respondent in answer to interrogatories to produce copies of written statements of prospective witnesses taken by its agents. In extending the Hickman rationale to include the attorney's agent, the court reasoned: " . . . we can see no logical basis for making any distinction between statements secured by a party's trial counsel and those obtained by others for the use of the party's trial counsel. In each case the statements are obtained in preparation for litigation and ultimately find their way into trial counsel's files for his use in representing his client at the trial." (*53*) Consequently, the Hickman protection has been extended to include agents other than the attorney who obtained statements for counsel's use (*54*).

Since 1950 and the extension made in the *Alltmont* case, the courts have utilized the work product concept of the *Hickman* case in dealing with land appraisers, their opinions and factual reports; in condemnation cases; and in dealing with expert testimony in general. A review of the condemnation cases of the past decade reveals a tendency of the courts to assume work product as a category and to deny or permit discovery on the basis of work product without explaining what is meant by the work product. Apparently the material sought will be considered work product, if it is shown that the material is of a legal or technical nature requiring the abilities of counsel, or an expert employed by counsel, to prepare the case in direct anticipation of litigation. Once the material sought is found to be work product, discovery will be denied unless factors of necessity are found to outweigh the merits of work product, and thus demand production of the material in the interest of justice (*55*).

Matter Subject to Discovery

Facts versus opinions

Many courts distinguish between factual and opinionative matter. As a result, the scope of examination has been circumscribed in Federal District Courts by judicial interpretation. Several States, however, have placed the opinion or conclusion of an expert beyond the bounds of discovery by enacting the proposed 1946 amendment to the Federal rules, (See p. 28). A tabulation of the status of the State laws is in appendix IV.

In the Federal District Courts, discovery in land acquisition cases is usually permitted of factual data but denied when opinionative matter is requested. For example, a Federal court observed that the reports of land appraisers included two types of information: (1) opinions of the appraisers, and (2) statements as to the factual bases upon which the opinions were predicated. The court then declared that the landowner might inspect, copy, or photograph the factual material but that the opinion material, to be determined by the court at an *in camera* inspection, would be withheld from the landowner (*56*).

Discovery has been denied, however, of not only the opinionative matter but also of the factual material contained in an appraiser's report (*57*). A court ruled that no special circumstances were present that justified an exception to the general rule as to the nondiscoverability of opinionative matter (*58*), and discovery of the facts of the appraisal report was denied on the basis that said facts were readily available to the landowner's appraisers. Accordingly, it was held in a recent case (*59*) that, without a showing of necessity, discovery would be limited to the facts upon which the opinions or conclusions were based but a liberal approach would be

used in determining what was fact and what was opinion.

In only one Federal case involving land acquisition, has the court ordered the production of appraisal reports for inspection and copying by the landowner without limiting discovery to factual material (*60*). The landowner's motion for production of the appraisal reports was granted, but the landowner was willing to pay part of the appraiser's expenses and neither the reports nor their authors were otherwise available. Upon a motion by the Government in this proceeding for an order limiting the matters to be inquired into in the taking of the deposition of the appraiser by oral examination, the court restricted the deposition to such matters as pertained to the fair market value of the subject matter of the litigation as of the date of taking and imposed no limitation upon the discovery of opinionative matter. This decision, however, has been distinguished (*61*), criticized (*62*), and questioned (*63*) in subsequent cases. In several other cases, the courts have followed similar rationale but have restricted discovery to factual data (*64*).

State courts have been much less inclined to make the fact-opinion distinction. An Iowa case (*65*) is the only example in which discovery of opinionative matter in a condemnation proceeding was denied. But here the conclusion of an expert was protected by the 1946 Amendment to the Federal rules, which Iowa had adopted. Virginia's Supreme Court of Appeals acknowledged that it at times had "made some distinction between the opinion of an expert and the evidence of a witness to facts." (*66*) However, this court permitted discovery of the appraiser's opinion on the gound that the appraiser was not the exclusive agent of the condemnor.

Yet, in a California condemnation case (*67*) a broad decision was made indicating that the appraiser's reports and their contents were within the attorney-client privilege (*68*). The privilege here did not extend to preclude the questioning of the expert as to his opinions and conclusions regarding the value of the lands and interest condemned, the reasons for the opinions, or to test the worth of the opinions by such inquiry on cross-examination as was relevant to the subject matter.

Expert opinions

In an even more recent California condemnation case (*69*), the court opined that material, whether factual or opinionative, is not privileged merely because it is the result of an expert's mental calculations, when the information on which it was predicated did not emanate from the attorney's client. Factual data is unprivileged because it did not emanate from the client, and an opinion formed by the expert thereon is similarly unprivileged. As a result of this decision, the reports and opinions of an appraiser are subject to discovery in California, because the appraiser would derive his information from an inspection of the land itself and not from the Government or condemning body.

Discovery of appraisal reports was permitted in Wisconsin though part of the file of the attorney general was prepared for litigation (*70*). The attorney-client privilege did not preclude the expert appraisers from disclosing any relevant opinions they had formed, whether reported or not. Neither were these opinions deemed to be protected from discovery as part of the work product of the attorney. Similarly, in Arizona, facts gathered by an adverse party's prospective witness and his opinion were subject to pretrial discovery (*71*). No validity was accorded the objection that the State was invading the work product of the landowner's attorney. The court said that the rules of civil procedure respecting discovery by interrogatories, fail to make any distinction between facts and opinions.

These cases indicate a tendency for some State courts to exclude opinion from the attorney-client privilege and work product categories of the Federal courts and leave opinionative material within the scope of examination. In only one case have Federal courts agreed with this trend and that has been distinguished as previously described. The Federal courts simply deny or permit discovery on the basis of whether the material sought is considered opinion or fact in the discretion of the court.

The confusion concerning the discoverability of expert opinion prompted Pennsylvania to amend its rules when contrary decisions were handed down in two cases that had similar factual patterns (*72*). The applicable rule was amended, effective April 1962, to read, "No discovery or inspection shall be permitted which . . . (f) would require a deponent, whether or not a party, to give an opinion as an expert witness, over his objection" (*73*). An explanatory note contains the statement that subdivision (f) does not attempt to define the difference between facts and opinion as an expert witness (*74*). This distinction must be made in each case. The amendment is also applicable in condemnation cases (*75*). Thus, in Pennsylvania, the opinions of an appraiser are protected from discovery before trial by an explicit statement in the rules permitting objection by the appraiser.

The Federal District courts have not permitted discovery of opinionative matter, whereas the State courts have been much more liberal in sanctioning such discovery. The recent holding in a New York District Court antitrust case may indicate a more liberal Federal approach, however. The court said: "Rather than impose an inflexible rule which would require laborious search for the intricate and elusive (and perhaps illusory) dividing markers separating fact, opinion, contention, and conclusion, it seems preferable to allow those interrogatories which might possibly call for opinion, conclusion or contention, if, on the balance of convenience, answers to them would serve any substantial purpose, either in leading to evidence or in narrowing issues" (*76*). This type of case can be distinguished from land acquisition cases by the complex factual situations involved in the antitrust cases.

Names of expert witnesses

In addition to the fact-opinion division, problems arise in connection with the discovery of appraisal reports and their preparation, names of the expert witnesses, the employer of the expert witnesses, the methods of appraisal, the qualifications of the appraiser and a breakdown of values. The Federal courts have consistently held in condemnation cases that the names and addresses of expert witnesses were not a proper subject of discovery (*77*). Nor may the agency or party for whom the appraisers made such reports be discovered (*78*). Discovery of the methods of appraisal used by the appraiser has been denied in two Federal court decisions (*79*), but in a 1963 decision in Louisiana, it was held that the State's witnesses would be required to answer all questions of fact asked in regard to their appraisal of the property and the method and manner used in making the appraisal (*80*). A U.S. District Court has held that the qualifications of the appraisers are not discoverable (*81*). Discovery of the specific values that the appraisers have placed on certain properties has likewise been denied in several cases (*82*).

Discovery of Government Information

A unique situation exists in the discovery of information against the Government, particularly in land acquisition cases where appraiser's reports, opinions, photographs, and statements are involved as provided in Federal rules 26(b), 33, 34, and 45. These rules apply to both the government as the condemnor and the private party. But the private parties are often concerned because of what seems to be the tendency against unnecessary disclosure of files of the executive branches of the Government. The disclosure of information in government files, however, must be evaluated in relation to the public interest and as to whether documents are of evidentiary value needed to obtain a just decision (*83*).

Disclosure of Government information has been permitted or denied on such considerations as: is the information necessary solely for the purpose of determining just compensation; is it privileged; is it fact or opinion; was it obtained in the ordinary course of business; is it the result of satisfying the requirements of Federal rules 26, 33, and 34; is it otherwise available; and was it obtained directly for the pending litigation. Among the kinds of information that have significance for discovery in right-of-way cases are documents; reports and statements, and expert materials. The status of the law of discovery with reference to these kinds of evidentiary matter is evaluated in the following paragraphs.

Documents, and Reports and Statements

A succinct summary of the law on documents, and reports and statements is in an analysis by Tolman: "There are now three

methods of obtaining documents from the adverse party before trial: by initial court order under rule 34; by interrogatories under rule 33 followed by court order to produce under rule 34; and by *subpoena duces tecum* at an oral deposition examination under rule 45, which, since 1946, does not specifically require initial court order and which commentators believe, and most courts hold, still should be construed to require it as to parties in order to provide consistency with rule 34. All of these procedures are subject to the protective control of the court on motion, under rule 30(b), of the party to whom the request for the document is directed." (*84*)

In permitting or denying discovery of Government documents and reports in condemnation cases, courts have considered the purpose for which the report was made; whether it was obtained in preparation for trial or in the ordinary course of business; whether good cause was shown in satisfaction of rule 34; whether the material sought is to be considered privileged; whether the information is essential to the litigation and otherwise unavailable; and whether the material sought requires discovery of fact or opinion.

When the issue was solely one of establishing a fair market value of the land at the time acquired by the condemnor, the burden to establish a fair value was on the defendant landowner, and the landowner was not entitled to the opinions or reports of the Government's expert appraisers in advance of trial (*85*).

A New York District Court condemnation proceeding in 1952 (*86*) permitted discovery of appraisal reports under certain limited conditions. The court stated: "It is shown that the appraisal reports in question were obtained by the Government for the express purpose of determining the compensation which would have to be paid for purchase of the property in question; that these reports are in possession and control of the government; and that neither the reports nor the authors thereof are otherwise available to the moving party. There is nothing to indicate that these reports can be regarded as privileged matter." (*87*)

In a 1959 proceeding in Ohio to condemn land for a ballistic missile launching site, discovery by interrogatories was permitted to ascertain the existence of documents supplied to appraisers, in order to determine the manner and criterion for valuation. But the discovery process did not apply to the valuation arrived at by the government's appraisers. The court followed the purpose test set out in *United States* v. *50.34 Acres of Land*, and said: ". . . while not wishing to express an opinion at this time on the extent to which the court in that case went—requiring disclosure of the final expert opinion—I think the principle is sound." (*88*) Discovery of certain statements and reports in a noncondemnation case in 1959 was permitted, the decision was based upon the following reasoning: "Statements or reports made in the ordinary course of business and not in preparation for trial do not embody the lawyers opinions, tactics, or conclusions, and accordingly they do not enjoy the privilege afforded the attorney's work." (*89*) The consensus is, however, that material gathered in preparation for trial either by the attorney or by someone retained by the attorney is generally considered a part of the attorney's work product and is not discoverable under the rationale of the *Hickman* and *Alltmont* cases.

The courts have been consistent in their requirement that the party seeking discovery satisfy the good cause requirement of rule 34. Discovery has been denied repeatedly by the courts when good cause was not shown (*90*). Production of documents from a party under rule 45 was denied when the moving party failed to satisfy the good cause requirement of rule 34. The court pointed out that the rules were to be construed *in pari materia* (*91*). This requirement, however, is not to be construed as a *fishing expedition*, the newer theory is that, it is more desirable to allow discovery of some immaterial facts than to deny discovery, which may bring to light facts that are material to the issue (*92*).

The question of privilege as it arises in land acquisition cases is usually related to the question of whether the documents sought are part of the attorney's work product. If the documents or papers sought are a part of the work product, they are considered privileged and not subject to discovery. If material sought to be discovered is in direct preparation for trial, and this material is essential to the litigation or the determination of the truth, and otherwise unobtainable, the courts may permit discovery (*93*). Where the production of transmittal letters was not apparently essential to the proper presentation of a taxpayer's suit, the court denied discovery, but cited Moore, on Federal Practice, as stating that: "It is a recognized general principle that in actions involving the administration of Federal law to which the Government is a party, production of government documents should be permitted unless 'the Court is satisfied that it would be against public policy to do so.' Moore, Federal Practice, 2d Ed., § 26.25(b), p. 1176" (*94*).

The courts have made it clear that discovery will be denied if unusual circumstances cannot be shown or if the material is otherwise available (*95*).

In summary, whether the documents or papers sought are fact or opinion relates directly to the problem of discoverability of expert testimony; and discovery of opinion material will be permitted only in special circumstances (*96*). The determination of special circumstances rests upon the judicial discretion of the courts.

Expert Testimony

In judicial proceedings conducted under the Federal rules the discovery of expert testimony arises from the provisions of rules 30, 31, and 33 as read in conjunction with rules 26 (a) and (b). Depositions may be filed to take the testimony of any person or party upon oral examination or written interrogatories for the purpose of discovery or for use as evidence, or for both purposes. With the exception of rule 33, all the rules provisions cited here provide for a party to take the deposition of any person. Rule 33, however, restricts written interrogatories to any adverse party. Although this provision limits the taking of depositions in a litigation, the process is less expensive and quicker than the provisions made by rules 30 and 31, and this provision has been more frequently used in cases involving land acquisition.

According to a 1962 Louisiana U.S. District Court decision (*97*), generally experts are immune from discovery (*98*); but many District Court case decisions have held that an expert's deposition may be taken and that a copy of his report is subject to discovery (*99*). When the taking of oral depositions of officers or agents of the United States who had knowledge of the value of the property involved was questioned, the court held that the owners were entitled to take the oral depositions with respect to facts but not with respect to opinions, and that a liberal approach was required to be adopted in determining what is fact and what is opinion (*100*). The U.S. District Court carefully distinguished its holding from that of *United States* v. *50.34 Acres of Land*, in which the same court 10 years earlier had permitted discovery of appraisal reports containing opinions within certain limitations; these limitations had been based primarily upon a showing of necessity.

The District Court of Rhode Island has held that a plaintiff was entitled to liberal discovery in attempting to ascertain facts surrounding methods employed by the defendant in production of its blood plasma, and the fact that the deponents possessed expert knowledge did not immunize them from examination (*101*). The court distinguished this case from the older case of *Lewis* v. *United Airlines Transport Corp.* (*102*), in that the plaintiff case was not taking the deposition of experts engaged by the defendant to make a study of the controversey. In the *Lewis* case an engineering expert had been employed by the defendant's attorney to assist in preparation for trial, and the court held that the expert was not required to disclose communications with his client nor answer questions calling for his expert opinion. In addition, a recent State case (*103*) discounts the validity of the *Lewis* case and all condemnation cases prior to Aug. 1, 1951, when the Federal rules were made applicable to condemnation proceedings (*104*). And in other Federal cases in which discovery of expert testimony has been denied, in both condemnation and noncondemnation proceedings, the courts based their holdings on some distinction between fact and opinion material, the work product privilege, the necessity of production, and whether the material was otherwise available (*105*).

Answers for which expressions of opinion were required that might later be used against a United States agency were not permitted. Other information not permitted was the names of witnesses and persons, which was part of the work product of the agency's attorney and otherwise available, *United States ex rel. TVA* v. *Bennett*, 14 F.R.D. 166

(E.D. Tenn. 1953). Despite the burden that a landowner has to establish the market value at the time the condemnor took the land, discovery of the expert's opinions or reports was denied. The majority view of the Federal courts with regard to condemnation cases is that the movant must be able to show some special circumstances that, in the interest of justice, require discovery and lift the material out of the privileged categories (*106*). The State courts generally adhere to the requirement of a showing of special circumstances to justify discovery, but they have been more liberal in permitting discovery than have the Federal courts, and they have adopted additional tests that place a great deal of discretionary power in the courts.

Procedural Innovations

Effective Sept. 1, 1963, New York adopted new procedural rules (*107*) that differ from the proposals of the Federal Rules Advisory Committee and from the existing rules in many respects. The new rules in New York, however, do not expand the scope or methods of discovery or provide for pretrial conferences in New York. Yet in a 1954 condemnation proceeding by a gas company, the court held that pretrial examinations in condemnation proceedings were consistent with the existing Practice Act on the grounds that a condemnation proceeding is a special proceeding within the meaning of article 29 of the Civil Practice Act, and that pretrial examination of the adverse party should be permitted (*108*).

Generally speaking New York courts have permitted discovery of pertinent information in condemnation proceedings when the issue was one of determining fair market value or just compensation (*109*). In a condemnation proceeding in which the claimant sought to examine the State's land and claim adjusters upon their appraisal of property (*110*), however, the court denied discovery and ruled that the materials sought were confidential, of an investigatory nature, and an essential part of the work product prepared in anticipation of litigation. Although the new rules establish the work product test and the anticipation of litigation test to deny discovery, the determination as to whether specified materials fall within these categories remains in the discretion of the court.

Discovery in California

California has had its own system of code pleading and practice for more than a century. The influence of the Federal rules on the California code has been reciprocal. In 1957 California added a new section to its code on depositions and discovery (*111*). The condemnation cases that have been litigated since then have reflected a continuing liberalization in permitting more discovery. In 1959, the California District Court of Appeals (*112*) held that an interrogatory seeking facts relevant to the issues and resting within the knowledge of the State, such as what acreage the State had already acquired in the same general area for similar purposes, would be permitted. Interrogatories calling for the names and addresses of the condemnor's appraisers and the contents of the appraiser's reports were ruled to be within the attorney-client privilege, an absolute bar, and were denied. This decision was broadened in 1962 by distinguishing between the appraiser's reports and contents and the appraiser's opinions and conclusions regarding the value of certain lands and severance damages. The court held that the attorney-client privilege did not extend to preclude questioning of the appraiser as to (1) his opinions and conclusions regarding the value of the lands and interest therein condemned, (2) severance damages, (3) special benefits, (4) and the reasons for his opinions. Permission was also given to test the worth of the appraiser's opinions by inquiry on cross-examination relevant to the subject matter (*113*). In July 1962, the Supreme Court of California went a step further in, *Oceanside Union School Dist.* v. *Superior Court* (*114*), holding that even the appraiser's reports were not within the attorney-client privilege and that their divulgence could be directed. The court based its reasoning upon the dominant purpose test (*115*) and whether the material sought emanated from the client. Thus the attorney-client privilege became qualified and no longer absolutely protected in California.

Florida

New rules were adopted in Florida in 1954, based primarily on the Federal rules. In this context, Florida endorsed the work product immunity of *Hickman* v. *Taylor* in *State Road Dept.* v. *Cline* (*116*), a 1960 case, in which the taking of depositions of three of the condemnor's appraisers was denied on the ground that the information sought was the work product of the condemnor and not subject to discovery. In 1961, however, the State Supreme Court reversed the 1960 District Court (*117*) in *Shell* v. *State Road Dept.* (*118*), and permitted the condemnees to inspect the appraiser's work sheets of the State Road Department. This case has particular significance because appraisal work sheets were involved. The court was very careful in distinguishing this condemnation case from an ordinary case. In the ordinary situation, the court said that the appraiser's work sheets would constitute a part of the work product but because one party was a government unit, it ruled otherwise. Unlike litigation between private parties, condemnation by any government authority, the court ruled, would place the condemnee at a disadvantage because the government has unlimited resources to which the condemnee contributes as a taxpayer (*119*).

Louisiana

Louisiana, traditionally a code State, adopted a new code effective Jan. 1, 1961, which exhibited an effort to consolidate and retain the basic Louisiana procedure of the older codes (1870) and to draw upon the Federal rules. The *State Department of Highway* v. *Buckman* case (*120*), involving highway condemnation, was litigated in 1960 prior to the effective date of the new rules. The court held that to require the State to produce certain written contracts and instructions concerning their appraisers was an error in the absence of any showing of undue hardship or injustice in the denial of production. Yet under the new rules, the court has permitted discovery by requiring plaintiff's experts to answer interrogatories regarding facts on which the appraisal of property was based and to have written memoranda available to help answer questions (*121*). This case under the new rules was distinguished from the 1960 case, in that the court did not require production or inspection of memoranda or written contracts but did require questions as to facts on which the appraisals were based. In 1963, the Court of Appeals carried the distinction between fact questions, opinions, and the written report or document a step further in *State* v. *Riverside Realty Co.* (*122*). It held that witnesses for the State would be required to answer all questions of fact asked in regard to their appraisal of the property and the method and manner used in making their appraisal, and that they would not be excused from answering such questions on the basis that they had to refer to written memoranda.

Virginia and other States

The Virginia Supreme Court of Appeals approached the problem somewhat differently and used an agency principle to permit discovery of the opinion of the condemnor's appraiser (*123*). Discovery was permitted as the appraiser was not the exclusive agent of the condemnor in the case, and the court pointed out that some distinction is made between fact and opinion evidence.

Wisconsin courts have also permitted discovery of the expert's relevant opinions and observations on the value of property upon the theory that such information was neither a part of the work product of the attorney general nor within the attorney-client privilege, but that a direct communication between the expert and his client or staff in connection with the condemnation of the property could be privileged (*124*).

In other States, such as Idaho, Illinois, Missouri, and Pennsylvania, courts have denied discovery following the reasoning of the Federal courts (*125*) and by emphasizing modified versions of Federal tests (*126*).

Proposed 1946 Amendment to Federal Rules and State Adoption

In 1944, the Advisory Committee to the Supreme Court began deliberations on possible amendments to the *Federal Rules of Civil Procedure*. Just before the Committee was to make its recommendations, the Supreme Court of the United States granted a writ of certiorari in the *Hickman* v. *Taylor* case. In that opinion, the Supreme Court adopted some of the Advisory Committee's proposals but rejected the proposal to amend rule 26(b) to include the discovery of data prepared for trial, and the conclusions of an attorney or expert. Appendix V includes examples of

typical amendments. Instead, the decision in Hickman was handed down to accomplish a similar result. Many States, however, have adopted the amendment in one form or another, affixing it either to their comparable rules 26(b), 30(b), 34, or as in one State, making it a separate rule in itself.

As proposed to the Supreme Court, the amendment has been adopted in substantially the same form by Idaho, Iowa, Kentucky, Louisiana, Nevada, New Jersey, Utah, and West Virginia. The effect has been to clothe any writing prepared in anticipation of litigation or in preparation for trial with a qualified privilege from discovery. In other words, the States extend the rationale in *Hickman* v. *Taylor* and grant a qualified immunization from discovery. Only upon a showing of undue hardship or injustice, attributable to the denial of discovery of such material, will discovery be permitted. A second effect of this rule is to grant an absolute immunity from discovery to any writing that reflects either an attorney's or an expert's conclusions. Under no circumstances is any written matter containing such an opinion subject to discovery.

In the jurisdictions that have adopted the rule, opinion is by statute eliminated from the proper scope of examination, although no clear differentiation has been made between fact and opinion. But inasmuch as an appraisal report, if prepared in anticipation of litigation, is qualifiedly protected, even the factual material is not per se subject to discovery. The result in these jurisdictions is a substantial degree of immunity from discovery for both the appraiser and the appraisal report.

In the case in Idaho, condemnees tried to get the appraisal reports and were unsuccessful because the appraisers were experts within the rule thus making conclusions of experts exempt from pretrial discovery (*127*). Similarly, an Iowa court observed that a writing containing the conclusion of an expert need not be produced for an adversary (*128*). Some State law is not so clear. For example, Louisiana has both permitted and denied discovery of appraisal reports by statutory interpretation of their comparable provision (*129*). A New Jersey court, in a personal injury action, expressly immunized from production or inspection the conclusions of an expert (*130*).

Several other States extend even greater protection to the expert and his work. Minnesota and Missouri, for example, have enacted the amendment with the exception of the qualification placed on the discovery of any writing prepared in anticipation of litigation (*131*). Consequently, the work product of an attorney is given the same absolute immunity from discovery as the conclusion of an attorney or expert. In a Missouri court, however, letters, memoranda, or notes prepared by appraisers for the highway department were exempted from discovery as a work product prepared in anticipation of litigation (*132*).

Illinois courts have likewise granted absolute immunity to reports or documents made in preparation for trial, although its new rule does not follow the proposed Federal amendment or the essentiality test of *Hickman* v. *Taylor* (*133*). The Pennsylvania Rules of Court also grant absolute immunity to both the reports prepared in anticipation of litigation (*134*), and to the opinions of expert witnesses, rules 4011 (d), (f).

In Texas, rule 167 of Rules of Civil Procedure grants absolute immunity to the communications involving the parties to the suit when it is "made in connection with the prosecution, investigation or defense" of a claim or the circumstances out of which the claim arose. No mention is made in the statute, however, of protection for the opinions and conclusions of either an attorney or an expert.

The language of the Washington rule 26(b) is unclear as to the type of protection given to the writing prepared in anticipation of litigation. Absolute immunity from discovery is accorded the conclusions of an attorney or an expert; but for the work product, instead of conditioning discovery on the showing of an injustice or undue hardship, the rule merely says, "The court need not order the production or inspection of any writing obtained or prepared . . ."

Maryland has enacted a rule, described in appendix V, with provisions similar to those of the proposed Federal rule. The result in Maryland, however, has been exactly opposite to that in the other 14 States. Instead of protecting the deponent from discovery, the Maryland rules offer him little or no opportunity to avoid disclosing all that he knows concerning the pending action. An appraiser's report is not protected even though it may have been obtained in anticipation of litigation. Only the mental impressions, conclusions, opinions, or legal theories of an attorney contained in a written report are protected. If no report has been written by the expert, he may then be examined by either oral or written deposition as to both his findings of fact and his opinions based thereon. To further emphasize the liberality of the discovery procedures, rule 406(b) states: "The policy of these Rules is to require full disclosure as specified in Rule 410 (Scope of Examination) and the powers conferred by section (a) of this Rule [providing for Orders to Protect Party and Deponent] shall be used only to prevent genuine oppression or abuse."

New York has enacted a modified version of the amendment, effective Sept. 1, 1963. Under the new Civil Practice Law and Rules, the work product of an attorney is not obtainable at all, according to § 3101(c). With regard to the material prepared for litigation, part (d) of the rule reads: "The following shall not be obtainable unless the court finds that the material can no longer be duplicated because of a change in conditions and that withholding it will result in injustice or undue hardship: (1) any opinion of an expert prepared for litigation; and (2) any writing or anything created by or for a party or his agent in preparation for litigation." Hence, the writing and opinion of an appraiser will be only qualifiedly protected from discovery; whereas, the work product of an attorney will enjoy an absolute protection. Consequently, New York has established another method of dealing with this problem, one that gives the appraiser less immunity from discovery than in most of the other States.

Thus, the proposed 1946 Amendment to Federal rules has had greatly different effects in the States. These effects varied from no protection at all for the reports and opinions of an appraiser, as in Maryland, to absolute immunity from discovery of both, as in Minnesota and Missouri. These differences follow the usual development of the law in which similar rules may be interpreted differently because of the historical development of the particular State and its legal needs.

Implications of Discovery in Land Acquisition Proceedings

Research findings indicate a tendency toward liberalizing discovery procedures in eminent domain proceedings. The influence of Federal and State rules on each other has resulted in more open procedures; consequently, the adversary content of a trial will be minimized by the exchange of information permitted under the rules. The Federal courts, especially, have made distinctions between the discovery of facts as opposed to the discovery of opinions. State courts, however, have been permitting the discovery of opinionative material in some recent cases, although there are qualifications as to how and under what circumstances discovery may be accomplished. Discovery of opinionative material, in many cases, is dependent upon the courts' discretion in weighing the private necessities against the public interest for justice, a philosophical basis that has had the result of making the discovery procedure applicable to new kinds of information.

In recent legislation several States have distinguished fact and opinion matter from the attorney's work product. In some States, fact as well as opinion material has been specifically excluded from discovery, but other States have included both types of information as being discoverable. A fair balance between the interests involved has been sought in the process of judicial construction—a balance between the individual and the sovereign. Although there have been a few cases in which it has been stated that, when a government unit is involved greater discovery against such a unit should be allowed. The courts have generally tended to treat litigants alike; as though both were private parties.

In eminent domain proceedings, public authorities can draw certain implications from the Federal and State trends based upon case and statute law. There is a trend toward discovery of appraisal work sheets; interrogatories concerning factual information regarding individual appraisals; and witnesses' names, including expert witnesses. Both public and private litigants no doubt produce preliminary papers of various sorts, which are in an incompleted stage. The courts, to avoid unnecessary interference with ordinary delib-

erations on the part of the experts involved, need to draw a distinction between reliable factual materials, which can be relied on by the movant in a case, and between different preliminary data that are not yet reliable information and are in the early stages of preparation. Although it would appear to be sensible to have expert opinion available to both sides prior to trial (to facilitate adequate trial preparation), many States and Federal Courts have regarded discovery of expert opinion as an invasion of the work efforts of the attorney.

Any efforts to reduce the adversary content of a trial and to arrive at a more factual approach to determination of issue would appear to be commendable. On the other hand, every effort should be made to eliminate a *fishing expedition* because of inadequate trial preparation on the part of the litigants, especially where the materials are readily available to both parties through the use of effort and imagination.

Federal, State, and private litigants must make certain that expert materials developed in anticipation of trial are well-prepared, well-documented, well-reasoned; that the factual materials are substantial; and that, if the data are discoverable, they are technically sound. With such efforts there need be less concern for discovery even where opinionative matter may be permitted. Appendixes furnish State citations for such items as (1) adoption of Federal rules by the State, and differences where they occur; (2) the adoption of the proposed 1946 Amendment to the Federal Rules by each State; and (3) the case and statute law on whether reports, conclusions, or work product are discoverable.

Wise application of the discovery rules to individual cases by the courts will provide the means whereby adequate information is made accessible to both parties and settlement of land acquisition cases is expedited. It may appear on the surface that the courts are moving with great haste toward use of discovery procedures, especially against experts, but examination in depth of the case law indicates that the courts offer considerable restraints against such indiscriminate use. Where such indiscriminate application has occurred, the legislatures have specified restraints. The development of discovery doctrines has a long history and has only recently affected the field of right-of-way litigation to any great extent. As other substantive fields of law have learned to adapt new procedures, so a compatibility of discovery procedures with eminent domain law will be developed. The greatest defense against inordinate use of the discovery process is well-prepared documents, papers, and other materials by the litigants involved. When careful preparation has been made use of the discovery process will permit the court to determine fair compensation to the condemnee based upon the adequacy of the information available to both parties.

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(*3*) *Economic Evidence in Right-of-Way Litigation*, by Sidney Goldstein, PUBLIC ROADS, vol. 32, No. 2, June 1962, pp. 21-39.

(*4*) 27 C.J.S. §1, 1959.

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(*9*) 1 Barron & Holtzoff, Federal Practice & Procedure, §9.33 (1962 Pocket Part).

(*10*) *Pretrial Practices in Condemnation Cases*, by D. R. Levin, Legal Affairs Committee annual meeting, AASHO, Dec. 1960, and *The New Rules in Arizona*, by L. R. Allen, 16 F.R.D. 191 (1955).

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(*18*) *State ex rel. Willey* v. *Whitman*, 91 Ariz. 120, 370 P. 2d 273 (Sup. Ct. 1962).

(*19*) Alaska R. Ct. P. & Admin. 72(a) (1963); Calif. R. P. §1262; Del. Code A., Sup. Ct. R. Civil, 71A (Cum. P.P. 1962); K.R.S. §177.081 (4) (effective June 19, 1952); Md. R. Civ. P., Rnle U12 (unannotated ed. 1963); and 4 Mo. R. S. 1959, Sup. Ct. R. 86.01.

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(*22*) *Aktiebalaget Vargas* v. *Clark*, 8 F.R.D. 635 (D.D.C. 1949); *Fidelis Fisheries* v. *Thorden*, 12 F.R.D. 179 (S.D. N.Y. 1952); and *Frankel* v. *Sussex Poultry Co.*, 71 A. 2d 754 (Super. Ct. Del. 1950).

(*23*) *Aktiebalaget Vargas* v. *Clark*, 8 F.R.D. 635 (D.D.C. 1949).

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(*46*) Reference *6* at p. 1032.

(*47*) *Hickman* v. *Taylor*, 329 U.S. 495 (1947).

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(*49*) Reference *35* at pp. 511, 512.

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(*59*) *United States* v. *284,392 Square Feet of Floor Space* 203 F. Supp. 75 (E.D.N.Y. 1962).

(*60*) *United States* v. *50.34 Acres of Land*, 13 F.R.D. 19 (E.D.N.Y. 1952).

(*61*) In *United States* v. *Certain Parcels of Land*, 25 F.R.D. 192 (N.D. Cal. 1959), the court decided that discovery of opinion had been permitted because of a showing of "compelling reasons." In *United States* v. *900.57 Acres of Land*, 30 F.R.D. 512 (W.D. Ark. 1962), the court decided that discovery of opinion had been permitted on the basis of the "extraordinary facts" of the case.

In *United States* v. *19.897 Acres of Land*, 27 F.R.D. 426 (E.D.N.Y. 1961) the same court said that it "cannot concur" in the opinion of *United States* v. *50.34 Acres of Land*.

(*63*) In *United States* v. *284,392 Square Feet of Floor Space*, 203 F. Supp. 75 (E.D.N.Y. 1962), decided by the same court, it was held that the decision in *United States* v. *50.34 Acres of Land* was "not in accordance with the most accepted authorities."

(*64*) *United States ex rel. TVA* v. *Bennett*, 14 F.R.D. 166 (F.D. Tenn. 1953); *Hickey* v. *United States*, 18 F.R.D. 88 (E.D. Pa. 1952); *United States* v. *900.57 Acres of Land*, 30 F.R.D. 512 (W.D. Ark. 1962).

(*65*) *Bryan* v. *Iowa State Hwy. Comm'n*, 251 Iowa 1093, 104 N.W. 2d 562 (1960); Code of Iowa 1962, R. Civ. P., 141 (a).

(*66*) *Cooper* v. *Norfolk Redevelopment Housing Authority*, 197 Va. 653, 90 S.E. 2d 788 (Sup. Ct. App. 1956).

(*67*) *Mowry* v. *Superior Court*, 20 Cal. Rptr. 698 (Dist. Ct. App. 1962).

(*68*) This part of the case was overruled by *San Diego Professional Ass'n* v. *Superior Court*, 23 Cal. Rptr. 384, 373 P. 2d 488 (1962).

(69) *Oceanside Union School Dist.* v. *Superior Court*, 23 Cal. Rptr. 375, 373 P. 2d 439 (1962).

(70) *State ex rel. Reynolds* v. *Circuit Court*, 15 Wis. 2d 311, 112 N.W. 2d 686 (1961).

(71) Reference *18*.

(72) *Straub* v. *Silver*, 22 D. & C. 2d 36 (1961), permitted unlimited examination of the opposing party's expert witness on the ground that the unamended rule permitted examination of "any matter, not privileged." *Wright* v. *P.T.C.*, 24 D. & C. 2d 334 (1961), arrived at the opposite decision.

(73) Pa. Rules of Court, Rule 4011(f), effective April 1962.

(74) *The April 1962, Amendments to the Pennsylvania Rules of Civil Procedure*, by R. W. Amram and S. Schulman, 33 Pa. B.A.Q. (1962).

(75) Reference *74*.

(76) *United States* v. *Renault, Inc.*, 27 F.R.D. 23 (S.D.N.Y. 1960).

(77) *Hickey* v. *United States*, 18 F.R.D. 88 (E.D. Pa. 1952); *United States ex rel. TVA* v. *Bennett*, 14 F.R.D. 166 (E.D. Tenn. 1953); *United States* v. *7534.04 Acres of Land*, 18 F.R.D. 146(N.D. Ga. 1954); *United States* v. *6.82 Acres of Land*, 18 F.R.D. 195 (D.N.M. 1955); *United States* v. *19.897 Acres of Land*, 27 F.R.D. 420 (E.D.N.Y. 1961); *United States* v. *900.57 Acres of Land*, 30 F.R.D. 512 (W.D. Ark. 1962).

(78) *Hickey* v. *United States*, 18 F.R.D. 88 (E.D. Pa. 1952); *United States* v. *19.897 Acres of Land*, 27 F.R.D. 420 (E.D. N.Y. 1961).

(79) *United States* v. *7534.04 Acres of Land*, 18 F.R.D. 146 (N.D. Ga. 1954); *United States* v. *6.82 Acres of Land*, 18 F.R.D. 195 (D.N.M. 1955).

(80) *State* v. *Riverside Realty Co.*, 152 So. 2d 345 (Ct. App. La. 1963).

(81) *United States* v. *19.897 Acres of Land*, 27 F.R.D. 420 (E.D.N.Y. 1961).

(82) *United States* v. *7534.04 Acres of Land*, 18 F.R.D. 146 (N.D. Ga. 1954), *United States* v. *6.82 Acres of Land*, *18* F.R.D. 195 (D.N.M. 1955), *United States* v. *19.897 Acres of Land*, 27 F.R.D. 420 (E.D.N.Y. 1961).

(83) *Reynolds* v. *United States*, 192 F. 2d 987,995 (1951), reversed on other grounds, 345 U.S. 1, 73 Sup. Ct. 528 (1953); and *United States* v. *Certain Parcels of Land Etc.*, 15 F.R.D. 224 (S. D. Cal. 1954).

(84) *Discovery Under the Federal Rules: Production of Documents and the Work Product of the Lawyer*, by L. L. Tolman, 58 Colum. D. Rev. 498, 503 (1958).

(85) *United States* v. *900.57 Acres of Land, More or Less*, 30 F.R.D. 512 (W.D. Ark. 1962).

(86) *United States* v. *50.34 Acres of Land, More or Less*, 13 F.R.D. 19 (E.D.N.Y. 1952).

(87) Reference 8, at p. 21.

(88) *United States* v. *62.50 Acres of Land, Etc.*, 23 F.R.D. 287 (M.D. Ohio 1959).

(89) *United States* v. *Swift & Co.*, 24 F.R.D. 282 (N.D. Ill. 1959).

(90) *Carpenter-Trant Drilling Co.* v. *Magnolia Petroleum Corp.*, 23, F.R.D. 257 (D. Neb. 1959); *Michet* v. *Meier*, 8 F.R.D. 464 (W.D. Pa. 1948), "In allowing plaintiff's motion under Rule 34, the court lays down the following showing plaintiff must make: (1) that there is "good cause" for the production and inspection of the desired material, (2) material requested must be "designated" with reasonable definiteness and particularity, (3) tbat the material must not be privileged, (4) material must constitute or contain evidence relating to matters within the scope of the examination permitted by Rule 26(b), i.e., it must be "relevant to the subject matter involved in the pending action," (5) the material must be within the possession, custody or control of the party upon whom demand is made; *Snyder* v. *United States*, 20 F.R.D. 7 (E.D.N.Y. 1956); *United States* v. *6.82 Acres of Land*, 18 F.R.D. 195 (D.N.M. 1955); *United States* v. *Certain Acres of Land*, 18 F.R.D. 98 (M.D. Ga. 1955); *United States* v. *19.897 Acres of Land Etc.*, 27 F.R.D. 420 (E.D.N.Y. 1961).

(91) *United States* v. *6.82 Acres of Land*, 18 F.R.D. 195 (D.N.M. 1955).

(92) *Reed* v. *Swift & Co.*, 11 F.R.D. 273 (W.D. Mo. 1951), and *The New Rules of Arizona*, by L. R. Allen, 16 F.R.D. 189 (1955).

(93) *Sachs* v. *Aluminum Company of America*, 167 F. 2d 571 (6 Cir. 1948), and *United States* v. *Certain Parcels of Land Etc.*, 15 F.R.D. 224 (D.S.D. Cal. 1954).

(94) *E. W. Bliss Co.* v. *United States*, 203 F. Supp. 175 (N.D. Ohio 1961).

(95) *United States* v. *Deere & Company*, 9 F.R.D. 523 (D. Minn. 1949); *United States* v. *7534.04 Acres of Land Etc.*, 18 F.R.D. 146 (N.D. Ga. 1954); aud *United States* v. *6.82 Acres of Land, More or Less*, 18 F.R.D. 195 (D.N.M. 1955).

(96) *United States* v. *Certain Parcels of Land Etc.*, 15 F.R.D. 224 (S.D. Cal. 1954); *United States* v. *Certain Parcels of Land*, 25 F.R.D. 192 (S.D. Cal. 1959); *United States* v. *4.724 Acres of Land, More or Less*, 31 F.R.D. 290 (E.D. La. 1962); and 4 Moore, Federal Practice, §26.24 at 1152, and specifically footnote 6 for case citations, and Supp. 1961, p. 81 (2d ed. 1953).

(97) *Maginnis* v. *Westinghouse Electric Corp.*, 207 F. Supp. 739 (E.D. La. 1962).

(98) *Lewis* v. *United Airlines Transport*, 32 F. Supp. 21 (W.D. Pa. 1940), engineering consultants' report on barrell of aircraft cylinder; *Boynton* v. *R. J. Reynolds Tobacco Co.*, 36 F. Supp. 593 (Mass. 1941), physician; *United States* v. *88 Cases, Etc.*, 5 F.R.D. 503 (D.N.J. 1946), chemists' analysis of orange beverage; *United States* v. *720 Bottles, Etc.*, 3 F.R.D. 466 (E.D.N.Y. 1946), chemists' analysis of vanilla extract; *Moran* v. *Pittsburgh-Des Moines Steele Co.*, 6 F.R.D. 594 (W.D. Pa. 1947), civil engineer's design on cylindrical liquefied gas tank, no expert involved, but in dicta held that experts may not be deposed; *Cold Metal Process Co.* v. *Aluminum Co.*, 7 F.R.D. 684 (Mass. 1947), metallurgist; *Roberson* v, *Graham Corp.*, 14 F.R.D. 83 (Mass. 1952), experts on antiques; *United States* v. *Certain Acres of Land, Etc.*, 18 F.R.D. 98 (M.D. Ga. 1955), expert land appraisers; *Colonial Airlines Inc.* v. *Jonas*, 13 F.R.D. 199 (S.D.N.Y. 1952), public accounts; *United States* v. *7534.04 Acres of Land, Etc.*, 18 F.R.D. 146 (N.D. Ga. 1954), expert land appraisers.

(99) *Bergstrom Paper Co.* v. *Continental Bus Co.*, 7 F.R.D. 548 (E.D. Wis. 1947), engineer's report on situs of explosion; *Cold Metal Process Co.* v. *Aluminum Co.*, 7 F.R.D. 425 (N.D. Ohio 1947), metallurgist report; *Sachs* v. *Aluminum Co.*, 167 F. 2d 570 (6 Cir. 1948), metallurgist; *Broadway & Ninety-Sixth St., Realty Co.* v. *Loew's Inc.*, 21 F.R.D. 347 (S.D.N.Y. 1958), expert in field of motion picture exhibition; *United States* v. *50.34 Acres of Land, Etc.*, 13 F.R.D. 19 (E.D.N.Y. 1952), expert land appraisers; *United States* v. *Certain Parcels of Land*, 15 F.R.D. 224 (S.D. Cal. 1953), expert land appraisers; *Watsh* v. *Reynolds Metals Co.*, 15 F.R.D. 376 (D.N.J. 1954), heating experts report; *Russo* v. *Merck & Co.*, 21 F.R.D. 237 (D.R.I. 1957), experts on production of blood plasma; *Cotden* v. *R. J. Schofield Motors*, 14 F.R.D. 521 (N.D. Ohio 1952), automotive expert's report.

(100) *United States* v. *284,392 Square Feet of Floor Space, Etc.*, 203 F. Supp. 75 (E.D.N.Y. 1962).

(101) *Russo* v. *Merck & Company*, 21 F.R.D. 237 (D.R.I. 1957).

(102) 32 F. Supp. 21 (W.D. Pa. 1940).

(103) See reference *18*.

(104) Fed. R. Civ. P., Rule 71A(a) (1951).

(105) *Hickey* v. *United States*, 18 F.R.D. 88 (E.D. Pa. 1952). *United States ex rel. TVA* v. *Bennett*, 14 F.R.D. 166 (E.D. Tenn. 1953); *United States* v. *6.8 Acres of Land, More or Less*, 18 F.R.D. 195 (D.N.M. 1955); *United States* v. *900.57 Acres of Land, More or Less*, 30 F. R.D. 512 (W.D. Ark. 1962).

(106) See reference *85*.

(107) N.Y. Civil Practice Laws & Rules, 185th Sess. 1962, C. 308, effective Sept. 1, 1963.

(108) *Algonquin Gas Transmission Co.* v. *Schwartz*, 132 N.Y.S. 2d 639, 206 Misc. 437 (Sup. Ct., Rocbland County 1954).

(109) *In re Union Turnpike*, 268 N.Y. 681, 198 N.E. 556 (Ct. App. N.Y. 1935); *In re Cross-Bronx Expressway*, 82 N.Y.S. 2d 55, 195 Misc. 842 (Sup. Ct. Bronx County 1948); *Hewitt* v. *State*, 216 N.Y.S. 2d 615, 27 Misc. 2d 930 (Ct. of Claims 1960). *Power Authority* v. *Kochan*, 216 N.Y.S. 2d 8 (Sup. Ct. 1961).

(110) *Valley Stream Lawns, Inc.* v. *State*, 164 N.Y.S. 2d 482, 6 Misc. 2d 607 (Ct. of Claims 1957).

(111) Calif. Code of Civ. P., Article 3. Deposition and Discovery, added by Stats. 1957, C. 1904, p. 3322, § 3, operative Jan. 1, 1958.

(112) *Rust* v. *Roberts*, 341 P. 2d 46 (Dist. Ct. App. Cal. 1954).

(113) *Mowry* v. *Superior Court*, 20 Cal. Rptr. 698 (Dist. Ct. App. 1962).

(114) 23 Cal. Rptr. 375, 373 P. 2d 439 (Sup. Ct. 1962).

(115) Tbe "dominant purpose" test was first laid down in *Holm* v. *Superior Court*, 42 Calif. 2d 500, 267 P. 2d 1025, 268 P. 2d 722 (1954).

(116) 122 So. 2d 827 (Dist. Ct. of App. Fla. 1960).

(117) *State Road Dept.* v. *Shell*, 122 So. 2d 215 (Dist. Ct. of App. Fla. 1960).

(118) 135 So. 2d 857 (1961).

(119) *Shelt* v. *State Road Dept.*, 135 So. 2d 857 (Fla. 1961).

(120) *State Department of Highways* v. *Buckman*, 239 La. 872, 120 So. 2d 461 (Sup. Ct. Pa. 1960).

(121) *State Department of Highways* v. *Spruell*, 243 La. 202, 142 So. 2d 396 (Sup. Ct. La 1962).

(122) 152 So. 2d 345 (Ct. App. La. 1963).

(123) 197 Va. 653, 90 S.E. 2d 788 (Sup. Ct. App. Va. 1956).

(124) *State ex rel. Reynolds* v. *Circuit Court*, 15 Wis. 2d 311, 112 N.W. 2d 886 (1961).

(125) *City of Chicago* v. *Harrison-Halsted Building Corp.*, 11 Ill. 2d 431, 143 N.E. 2d 40 (1957); *State* v. *Bair*, 83 Ida. 478, 365 Pa. 2d 216 (1961); and *State ex rel. State Highway Comm'n.* v. *Jensen*, 362 S.W. 2d 568 (Mo. Sup. Ct. 1962).

(126) *Musulin* v. *Redevelopment Authority*, 25 Pa. County R. 267 (1961); and *Construction of Vine Street Extension*, 18 D. & C. 2d 115 (Pa. 1959).

(127) *State* v. *Bair*, 83 Ida. 478, 365 P. 2d 216 (1961).

(128) *Bryan* v. *Iowa State Hwy. Comm'n.*, 251 Iowa 1043, 104 N.W. 2d 562 (1960).

(129) *State Department of Highways* v. *Spruelt*, 243 La. 202, 142 So. 2d 396 (1962); and *State Department of Highways* v. *Buckman*, 239 La. 872, 120 So. 2d 461 (1960).

(130) *Cermak* v. *Hertz Corp.*, 53 N.J. Super. 455, 147 A. 2d 800 (1958).

(131) Minn. R. Civ. P., Rule 26.02; Mo. Sup. Ct. Rule 57.01(b).

(132) *State ex rel. State Hwy. Comm'n.* v. *Jensen*, 362 S.W. 2d 568 (Mo. 1962).

(133) Ill. Sup. Ct. Rule 19-5; Ill. Annot. Stat., § 101.19-5 (Supp. 1962).

(134) *Musulin* v. *Redevelopment Authority*, 25 D. & C. 2d 267 (Pa. 1961).

APPENDIX I

Federal Rules Pertinent to the Discovery Process

(Federal Rules of Civil Procedure, Title 28 U.S.C., as amended 1961)

Rule 16—Pretrial Procedure; Formulating Issues

In any action, the court may in its discretion direct the attorneys for the parties to appear before it for a conference to consider: (1) The simplification of the issues; (2) the necessity or desirability of amendments to the pleadings; (3) the possibility of obtaining admissions of fact and of documents which will avoid unnecessary proof; (4) the limitation of the number of expert witnesses; (5) the advisability of a preliminary reference of issues to a master for findings to be used as evidence when the trial is to be by jury; (6) such other matters as may aid in the disposition of the action.

The court shall make an order which recites the action taken at the conference, the amendments allowed to the pleadings, and the agreements made by the parties as to any of the matters considered, and limits the issues for trial to those not disposed of by admissions or agreements of counsel; and such order when entered controls the subsequent course of the action, unless modified at the trial to prevent manifest injustice. The court in its discretion may establish by rule a pretrial calendar on which actions may be placed for consideration as above provided and may either confine the calendar to jury actions or to nonjury actions or extend it to all actions.

Rule 26—Depositions Pending Action

(b) Scope of Examination. Unless otherwise ordered by the court as provided by rule 30 (b) or (d), the deponent may be examined regarding any matter, not privileged, which is relevant to the subject matter involved in the pending action whether it relates to the claim or defense of the examining party or to the claim or defense of any other party, including the existence, description, nature, custody, condition, and location of any books, documents, or other tangible things and the identity and location of persons having knowledge of relevant facts. It is not ground for objection that the testimony will be inadmissible at the trial if the testimony sought appears reasonably calculated to lead to the discovery of admissible evidence.

Rule 30—Depositions Upon Oral Examination

(b) Orders for the Protection of Parties and Deponents. After notice is served for taking a deposition by oral examination, upon motion seasonably made by any party or by the person to be examined and upon notice and for good cause shown, the court in which the action is pending may make an order that the deposition shall not be taken, or that it may be taken only at some designated place other than that stated in the notice, or that it may be taken only on written interrogatories, or that certain matters shall not be inquired into, or that the scope of the examination shall be limited to certain matters, or that the examination shall be held with no one present except the parties to the action and their officers or counsel, or that after being sealed the deposition shall be opened only by order of the court, or that secret processes, developments, or research need not be disclosed, or that the parties shall simultaneously file specified documents or information enclosed in sealed envelopes to be opened as directed by the court; or the court may make any other order which justice requires to protect the party or witness from annoyance, embarrassment, or oppression.

(d) Motion to Terminate or Limit Examination. At any time during the taking of the deposition, on motion of any party or of the deponent and upon a showing that the examination is being conducted in bad faith or in such manner as unreasonably to annoy, embarrass, or oppress the deponent or party, the court in which the action is pending or the court in the district where the deposition is being taken may order the officer conducting the examination to cease forthwith from taking the deposition, or may limit the scope and manner of the taking of the deposition as provided in subdivision (b). If the order made terminates the examination, it shall be resumed thereafter only upon the order of the court in which the action is pending. Upon demand of the objecting party or deponent, the taking of the deposition shall be suspended for the time necessary to make a motion for an order. In granting or refusing such order the court may impose upon either party or upon the witness the requirement to pay such costs or expenses as the court may deem reasonable.

Rule 31—Depositions of Witnesses Upon Written Interrogatories

(d) Orders for the Protection of Parties and Deponents. After the service of interrogatories and prior to the taking of the testimony of the deponent, the court in which the action is pending, on motion promptly made by a party or a deponent upon notice and good cause shown, may make any order specified in rule 30 which is appropriate and just or an order that the deposition shall not be taken before the officer designated in the notice or that it shall not be taken except upon oral examination.

Rule 33—Interrogatories to Parties

Any party may serve upon any adverse party written interrogatories to be answered by the party served or, if the party served is a public or private corporation or a partnership or association, by any officer or agent, who shall furnish such information as is available to the party. Interrogatories may be served after commencement of the action and without leave of court, except that, if service is made by the plaintiff within 10 days after such commencement, leave granted with or without notice must first be obtained. The interrogatories shall be answered separately and fully in writing under oath. The answers shall be signed by the person making them; and the party upon whom the interrogatories have been served shall serve a copy of the answers on the party submitting the interrogatories within 15 days after the service of the interrogatories, unless the court, on motion and notice and for good cause shown, enlarges or shortens the time. Within 10 days after service of interrogatories a party may serve written objections thereto together with a notice of hearing the objections at the earliest practicable time. Answers to interrogatories to which objection is made shall be deferred until the objections are determined.

Interrogatories may relate to any matters which can be inquired into under rule 26(b), and the answers may be used to the same extent as provided in rule 26(d) for the use of the deposition of a party. Interrogatories may be served after a deposition has been taken, and a deposition may be sought after interrogatories have been answered, but the court, on motion of the deponent or the party interrogated, may make such protective order as justice may require. The number of interrogatories or of sets of interrogatories to be served is not limited except as justice requires to protect the party from annoyance, expense, embarrassment, or oppression. The provisions of rule 30(b) are applicable for the protection of the party from whom answers to interrogatories are sought under this rule.

Rule 34—Discovery and Production of Documents and Things for Inspection, Copying, or Photographing

Upon motion of any party showing good cause therefor and upon notice to all other parties, and subject to the provisions of rule

30(b), the court in which an action is pending may (1) order any party to produce and permit the inspection and copying or photographing, by or on behalf of the moving party, of any designated documents, papers, books, accounts, letters, photographs, objects, or tangible things, not privileged, which constitute or contain evidence relating to any of the matters within the scope of the examination permitted by rule 26(b) and which are in his possession, custody, or control; or (2) order any party to permit entry upon designated land or other property in his possession or control for the purpose of inspecting, measuring, surveying, or photographing the property or any designated object or operation thereon within the scope of the examination permitted by rule 26(b). The order shall specify the time, place and manner of making the inspection and taking the copies and photographs and may prescribe such terms and conditions as are just.

Rule 45—Subpoena

(b) For Production of Documentary Evidence. A subpoena may also command the person to whom it is directed to produce the books, papers, documents, or tangible things designated therein; but the court, upon motion made promptly and in any event at or before the time specified in the subpoena for compliance therewith, may (1) quash or modify the subpoena if it is unreasonable and oppressive or (2) condition denial of the motion upon the advancement by the person in whose behalf the subpoena is issued of the reasonable cost of producing the books, papers, documents, or tangible things.

(d) Subpoena for Taking Depositions; Place of Examination. (1) Proof of service of a notice to take a deposition as provided in rules 30(a) and 31(a) constitutes a sufficient authorization for the issuance by the clerk of the district court for the district in which the deposition is to be taken of subpoenas for the persons named or described therein. The subpoena may command the person to whom it is directed to produce designated books, papers, documents, or tangible things which constitute or contain evidence relating to any of the matters within the scope of the examination permitted by rule 26(b), but in that event the subpoena will be subject to the provisions of subdivision (b) of rule 30 and subdivision (b) of this rule 45.

Rule 71A—Condemnation of Property

(a) Applicability of Other Rules. The Rules of Civil Procedure for the United States District Courts govern the procedure for the condemnation of real and personal property under the power of eminent domain, except as otherwise provided in this rule.

APPENDIX II

Expanded Version of Federal Pretrial Conference Provision

Code of Iowa 1962

Code of Iowa 1962, vol. II, Rules of Civil Procedure, § 136, Pretrial conference. After issues are joined, the court may in its discretion and shall on written request of any attorney in the case direct all attorneys in the action to appear before it for a conference to consider, so far as applicable to the particular case:

a. The necessity or desirability of amending pleadings by formal amendment or pretrial order.

b. Agreeing to admissions of fact, documents or records not really controverted, to avoid unnecessary proof.

c. Limiting the number of expert witnesses.

d. Settling any facts of which the court is to be asked to take judicial notice.

e. Stating and simplifying the factual and legal issues to be litigated.

f. Specifying all damage claims in detail as of the date of the conference.

g. All proposed exhibits and mortality tables and proof thereof.

h. Consolidation, separation for trial, and determination of points of law.

i. Questions relating to *voir dire* examination of jurors and selection of alternate jurors, to serve if a juror becomes incapacitated.

j. Possibility of settlement.

k. Filing of advance briefs when required.

l. Any other matter which may aid, expedite, or simplify the trial of any issue.

The pretrial judge may direct the parties to the action to be present or immediately available at the time of conference. (Report 1943, amendment 1961.)

APPENDIX III

Adoption of Federal Discovery Rules by States

The following table reflects a comparison of the various discovery and pretrial provisions of each state and the District of Columbia. An explanation of the symbols follows the table.

The table for Maine was compiled from the statutes available to the researcher. Maine, however, repealed these provisions by a 1959 statute and adopted new rules similar to the Federal Rules, effective Dec. 1, 1959.

Michigan, by order of the Michigan Supreme Court, adopted new rules effective Jan. 1, 1963. *Procedural Changes in Michigan,* by J. L. Honigman, 31 F.R.D. 113 (1962). Discovery provisions in the new rules have been changed to conform substantially to the Federal practice, excepting, however, that only evidence admissible at the trial may be taken in discovery proceedings. A copy of these rules was not available for an accurate comparison.

Those States which have few or no discovery provisions have been designated code States, in that their provisions show little or no Federal influence.

Statutory References by States

Alabama: Ala. Code, Recompiled 1958, Title 7 (1960).

Alaska: Alaska R. Ct. Proc. & Adm'n (1963).

Arizona: 16 Ariz. Rev. Stat. Ann., R. Civ. P. (1956).

Arkansas: 3A Ark. Stat. Ann. 1947, Title 28 (1962 Replacement).

California: 23 Wests' Ann. Code Div. P. (Cum. P.P. 1962).

Colorado: 1 Colo. Rev. Stats, Ch. 4 (1953).

Connecticut: Conn. Practice Book of 1951 (Cum. Supp. 1960).

Delaware: 13 Del. Code Ann., Super. Ct. R.—Civil (1953).

District of Columbia: Munic. Ct. R. (1961).

Florida: 30 Fla. Stat. Ann. (1954).

Georgia: 38 Ga. Code Ann. (1959) (Cum. P.P. 1961).

Hawaii: Hawaii R. Civ. P. (1954).

Idaho: 2 Idaho Code, R. Civ. P. (Cum. P. Supp. 1961).

Illinois: 110 Smith-Hurd Ill. Ann. Stat., Sup. Ct. R. (1956).

Indiana: 2 Burns Ind. Stat. Ann.; I.L..E Depositions and Discovery § 1 (Cum P.P. 1962).

Iowa: 2 Iowa Code 1962, R. Civ. P.

Kansas: Gen. Stat. Kan. Ann. § 60 (1949).

Kentucky: Ky. Rev. Stat., R. Civ. P. (1953).

Louisiana: 3,4 La. Stat. Ann. (1961).

Maine: Me. Rev. Stat. 1959 (Cum. Supp. 1961).

Maryland: Md. R. Civ. P. (1961 ed.), (unannot. ed. 1963).

Massachusetts: 38 Mass. Gen. L. Ann. (1960).

Michigan: Gen. Ct. R. of 1963.

Minnesota: 27A Minn. Stat. Ann., R. Civ. Dist. Ct. (1958).

Mississippi: 2 Miss. Code 1942 § 1699 (1957).

Missouri: 4 Mo. Rev. Stat., Sup. Ct. R. (1957).

Montana: Mont. Laws, 37th Sess. Ch. 13 (1961).

Nebraska: Neb. Sess. Laws 1951, §§ 25–1267.01–25.1269 (1952).

Nevada: 1 Nev. Rev. Stat., R. Civ. P. (1953).

New Hampshire: 5 N.H. Rev. Stat. Ann. §516 (1955).

New Jersey: N.J. Practice, Part IV, Ch. 4 (1953).

New Mexico: 4 N.M. Stat. Ann. 1953, Ch. 21 (1954).

New York: N.Y. Laws, Civ. Prac. L. & R., Ch. 308 (1962).

North Carolina: 1A N.C. Gen. Stat., Recompiled 1953.

North Dakota: 5 N.D. Cent. Code Ann., R. Civ. P. (1957).

Ohio: Ohio Rev. Code Ann., Ch. 2317 (1958) (1962 Supp.).

Oklahoma: Okla. Stat. Ann., Title 12, Ch. 10 (1960).

Oregon: 1 Ore. Rev. Stat., Ch. 45 (Ch. Replaced 1961–1962).

Pennsylvania: Pa. R. of Ct. 1962.

Rhode Island: 2 Gen. Laws R.I., Title 9, Ch. 18 (1962).

South Carolina: 6 Code Laws S.C., Ch. 7 (1962).

South Dakota: 2 S.D. Code, Title 36, Ch. 36 (1960 Supp.).

Tennessee: 5 Tenn. Code Ann., Title 24 (1955) (Supp. 1962).

Texas: Tex. R. Civ. P. (1955).

Utah: 9 Utah Code Ann., R. Civ. P. (1953).

Vermont: 3 Vt. Stat. Ann., Title 12 (1961 Supp.).

Virginia: 2 Va. Code, Title 8, R. Sup. Ct. App. (1950).

Table 1.—Adoption of Federal discovery rules by States

States	Rules citations									
	16	26(b)	30(b)	30(d)	31(d)	33	34	45(b)	45(d)	71A(a)
Alabama	N	F	F	N	N	I	N	I	N	N
Alaska	F	F	F	F	F	F	F	F	F	F
Arizona	F	F	F	F	F	F	F	F	F	N
Arkansas	F	F	F	F	S	F	F	S	S	N
California	FE	FA	FA	F	F	FC	F	S	S	S
Colorado	F	F	FA	F	F	F	F	F	F	N
Connecticut	I	I	S	N	S	S	I	I	N	N
Delaware	F	F	F	F	F	S	F	F	IF	IF
District of Columbia	S	F	F	F	F	F	F	F	N	N
Florida	S	F	F	F	S	S	F	F	F	N
Georgia	F	F	F	F	F	F	FA	I	F	N
Hawaii	F	F	F	F	F	F	F	F	F	N
Idaho	F	FA	F	F	F	FA	F	F	FE	N
Illinois	F	I	S	S	S	S	I	I	I	N
Indiana	S	S	I	I	I	I	I	I	N	N
Iowa	FE	S	FA	F	I	I	S	S	I	N
Kansas	F	I	N	N	N	N	I	N	I	N
Kentucky	F	F	FA	F	F	F	F	F	S	S
Louisiana	S	F	FA	F	F	S	F	S	S	N
Maine [2]	N	I	N	N	N	N	N	N	N	N
Maryland	FA	FA	FA	I	I	I	FA	S	F	IF
Massachusetts [1]	F	I				I	I			N
Michigan [3]										
Minnesota	F	FA	FA	F	F	FA	F	F	F	N
Mississippi [1]	N	I					I	N	N	N
Missouri	F	FA	S	S	F	FA	S	I	I	
Montana	F	F	F	F	F	F	F	F	S	N
Nebraska	N	S	F	F	F	FA	F	I	I	N
Nevada	F	F	F	F	F	F	F	F	F	N
New Hampshire [1]										
New Jersey	FE	FA	F	F	F	FE	F	I	N	N
New Mexico	F	F	FA	F	F	F	F	F	F	N
New York	N	I	I	N	I	N	I	I	N	N
North Carolina	IF	I	I	N	I	N	I	I	N	N
North Dakota	F	F	F	F	F	FA	FA	F	F	N
Ohio [1]	N	I					I	I	N	N
Oklahoma [1]							I			N
Oregon [1]	N		IF	S						N
Pennsylvania	S	I	I	I	N	S	I	I	N	N
Rhode Island [1]	N	I								N
South Carolina	N	I	I	N	I	N	I	I	N	N
South Dakota	F	I	F	F	F	I	I	F	F	N
Tennessee	N	S	FA	I	I	N	N	N	N	N
Texas	S	I	N	N	N	I	FA	I	N	N
Utah	F	F	FA	F	F	F	F	F	F	N
Vermont	S	F	F	F	S	I	S	N	F	N
Virginia	S	I					I	I	N	N
Washington	S	FA	F	F	F	F	F	N	N	N
West Virginia	F	IF	F	S	F	F	FA	F	F	N
Wisconsin	F	IF	S	N	S	N	I	N	N	IF
Wyoming	F	F	F	F	F	F	FA	F	F	N

Meaning of symbols is as follows:

F, Same as Federal rule.
FC, Federal rule changed.
FE, Federal rule expanded.
FA, Federal rule plus an additional paragraph.
I, Individual State rule.
IF, Individual State rule showing Federal influence.
N, No comparable rule.
S, Substantially the same as the Federal Rule.

[1] States showing little Federal influence.
[2] Data for Maine reflect the rules in effect before their 1959 repealing statute. More recent material was unavailable.
[3] Michigan adopted new rules substantially the same as the Federal rules, effective Jan. 1, 1963, but a copy of the new rules was not available for this study.

Washington: 0 Rev. Code Wash., R. Pleading, Prac. & P. (1960).

West Virginia: 3 W. Va. Code Ann., R. Civ. P. (1961).

Wisconsin: 30, 38 Wis. Stat. Ann. (1958).

Wyoming: 2 Wyo. Stat. 1957, R. Civ. P. (1959).

APPENDIX IV

Status of State Laws as to Privileged Matter

The following States have adopted the proposed 1946 amendment to the Federal rules regarding experts and attorney's work product: Idaho, Illinois, Iowa, Kentucky, Louisiana, Maryland, Minnesota, Nevada, New Jersey, New York, Pennsylvania, Texas, Utah, Washington, and West Virginia.

Expert Reports

The following States provide absolute protection to expert reports from the discovery procedures: Illinois, Minnesota, Missouri, Ohio, Pennsylvania, and Texas.

The following States provide no protection to expert reports from the discovery procedures: Arizona, California, Florida, Maryland, and Wisconsin.

The following States provide qualified protection to expert reports from the discovery procedures: Delaware, Idaho, Iowa, Kentucky, Louisiana, Mississippi, Nevada, New Hampshire, New Jersey, New York, Utah, and West Virginia.

Expert Conclusion

The following States provide absolute protection to expert conclusion from the discovery procedures: Idaho, Illinois, Iowa, Kentucky, Minnesota, Missouri, Nevada, New Jersey, Pennsylvania, Utah, Washington, and West Virginia.

Maryland provides no protection to expert conclusion from the discovery procedures.

The following States provide qualified protection to expert conclusion from the discovery procedure: Arkansas, Colorado, Florida, Louisiana, New York, Ohio, Virginia, and Wisconsin.

Work Product

The following States provide absolute protection to work products from the discovery procedures: Connecticut, Georgia, Idaho, Illinois, Iowa, Kentucky, Louisiana, Minnesota, Nevada, New Jersey, New York, Ohio, Pennsylvania, Texas, Utah, Washington, and West Virginia.

The following States provide qualified protection to work products from the discovery procedures: Arizona, California, Delaware, Florida, Maryland, Michigan, Missouri, New Hampshire, and Wisconsin.

Summary Explanation by State

Alabama

No cases on point, Code of Ala., Recompiled 1958, Title 7, §§474–489. Provisions are based on the Federal provisions for discovery, but no provision is made for production of documents, requests for admissions, written interrogatories or the other discovery devices available under the Federal rules. Rules similar to the Federal rules were proposed in 1957 but rejected in the Senate.

Alaska

No cases on point. Alaska R. of Ct. Proc. & Admin. 1963, Fed. R. Civ. P. made effective in Alaska on July 18, 1949; 63 Stat. 445, 48 U.S.C.A. §103a (1952).

Arizona

Rules virtually identical to Federal rules were adopted, effective Jan. 1, 1940. Latest revision effective Jan. 1, 1956. *Dean* v. *Superior Court*, 84 Ariz. 104, 324 P. 2d 764(1958), denying discovery of work product; *State ex rel. Willey* v. *Whitman*, 91 Ariz. 120, 370 P. 2d 273 (1962), condemnation case permitting discovery of reports and conclusions.

Arkansas

No cases on point. Procedure is regulated entirely by the legislature. 3A Ark. Stat. Ann. 1947, Title 28 (1962 Replacement). In 1949, provisions similar to Fed. R. 16 were adopted, Ark. Stat. §27–2401 (Supp. 1947). In 1953 legislature adopted provisions similar to Fed. Rules 26–37, Ark. Stat. §§28–347 to 28–361 (Supp. 1957). As to liberal construction of rules see, *Arkansas State Hwy. Comm'n* v. *Stanley*, 353 S. W. 2d 173 (1962).

California

Adopted code pleading in 1851. Judicial Council adopted rules similar to Federal pretrial and discovery provisions in 1957 and 1958, Cal. Stat. 1957 §3, e. 1904 p. 3322, operative Jan. 1, 1958. *Oceanside Union School Dist.* v. *Superior Court*, 23 Cal. Rptr. 375, 373 P. 2d 439 (1962), condemnation case permitting discovery of expert reports; *Greyhound Corp.* v. *Superior Court*, 56 C. 2d 355, 364 P. 2d 266 (1961), permitting discovery of work product; for recent discussion of scope of discoverability of expert reports, conclusions, and work product, see *Brown* v. *Superior Court*, 30 Cal. Rptr. 338 (Dist. Ct. App. 1963).

Colorado

No cases on point. Procedural rules have been similar in text and interpretation to Federal provisions since April 6, 1941. 1 Col. Rev. Stat., Ch. 4 (1953). Keely, *How Colorado Conformed State To Federal Civil Procedure*, 16 F.R.D. 291 (1955).

Connecticut

Adopted a code in 1879 based on the Field Code. In 1957 the Conn. Sup. Ct. adopted rules providing for limited disclosure and pretrial practice. Conn. Practice Book of 1951 (Cum. Supp. 1960). *Prizio* v. *Penachio*, 19 Conn. Sup. 381, 115 A. 2d 340 (Conn. Super. 1955), indicating a trend toward Federal interpretation but protecting written statements as work product.

Delaware

Adopted rules similar to Federal provisions, Effective Jan. 1, 1948, 13 Del. Code Ann., Super. Ct. R.—Civil (1953). *Empire Box Corp.* v. *Illinios Cereal Mills*, 90 A. 2d 672 (Super Ct. 1952), denying discovery and qualifying protection to expert reports and work product.

Florida

Rules adopted March 15, 1954, based primarily on the Federal rules. 30 Fla. Stat. Ann. (1954). *Shell* v. *State Road Dept.*, 155 So. 2d 857 (Fla. 1961), condemnation case permitting discovery of appraiser's work sheets; *Shawmut Van Lines Inc.* v. *Small*, 148 So. 2d 556 (Dist. Ct. App. Fla. 1963), noncondemnation case qualifying discovery of work product.

Georgia

As of March 25, 1959, the code of Georgia stands amended following for the most part the Federal discovery provisions. 38 Ga. Code Ann. (1959) (Cum. P.P. 1961). *Setzers Super Stores* v. *Higgins*, 104 Ga. App. 116, 121 S.E. 2d 305 (Ga. App. Ct. 1961), noncondemnation case denying discovery of work product.

Hawaii

No cases on point. Rules were adopted effective June 14, 1954, substantially the same as the Federal Rules. Hawaii R. Civ. P. 1954.

Idaho

Rules that follow closely the Federal rules were adopted effective Nov. 1, 1958. 2 Idaho Code, R. Civ. P. (Cum. P. Supp. 1961). *State* v. *Bair*, 83 Idaho 478, 365 P. 2d 216 (1961), condemnation case denying discovery of experts' conclusions.

Illinois

A new Civil Practice Act, influenced by Federal rules, was adopted, effective Jan. 1, 1956, 110 Smith—Hurd Ill. Ann. Stat., Sup. Ct. R. (1956). *City of Chicago* v. *Harrison-Halsted Bldg. Corp.*, 11 Ill. 2d 431, 143 N.E. 2d 40 (Sup. Ct. 1957), condemnation case denying discovery of experts' statements. *Kemeny* v. *Skorch*, 159 N.E. 2d 489, 490 (Ill. 1959), as to documents exempt from disclosure.

Indiana

No cases on point. Indiana has rules provisions similar to Federal provisions rules 16 and 26(b), but has its own limited provisions for depositions and discovery. 2 Burns Ind. Stat. Ann. (1947); I.L.E., Depositions & Discovery § 1 (Cum. P.P. 1962).

Iowa

Rules adopted effective July 4, 1943 are less liberal than the corresponding Federal provisions. 2 Iowa Code 1962, R. Civ. P., *Bryan* v. *Iowa State Hwy. Comm'n*, 251 Iowa 1093, 104 N.W. 2d 562 (1960), condemnation case in which discovery of experts' conclusions were denied; *Hanke* v. *Iowa Home Mut. Gas Co.*, 87 N.W. 2d 920 (1958), a noncondemnation case qualifying discovery of attorney's work product.

Kansas

No cases on point. Procedure to a great extent remains unchanged since 1859. Pretrial procedure corresponding to Fed. R. 16 was adopted effective June 30, 1949. Gen. Stat. Kan. Ann. § 60 (1949). *Pyramid Life Ins. Co.* v. *Gleason Hospital, Inc.*, 188 Kan. 95, 360 P. 2d 858 (1961), as to general interpretation of discovery statute.

Kentucky

Rules similar to the Federal rules were adopted effective July 1, 1953, Ky. Rev. Stat., R. Civ. P. (1953). *Bender* v. *Eaton*, 343 S.W. 2d 799 (Ky. 1961), a noncondemnation case denying discovery of work product.

Louisiana

Code revision became effective Jan. 1, 1961; 3, 4 La. Stat. Ann. (1961); *State Department of Hwys.* v. *Buckman*, 239 La. 872, 120 So. 2d 461 (1960), condemnation case denying discovery of certain contracts and instructions; *State* v. *Riverside Realty Co.*, 152 So. 2d 345 (Ct. App. La. 1963), condemnation case permitting discovery of expert factual questions without violating work product.

Maine

No cases on point. Rules similar to Federal rules were adopted effective Dec. 1, 1959. Me. Rev. Stat. 1959 (Cum. Supp. 1961).

Maryland

No cases on point. A complete revision of the rules was promulgated in 1956 and revised rules became effective Jan. 1, 1957. Md. R. Civ. P. (1961 ed.) and as amended through Sept. 1, 1963 (unannotated ed. 1963). *Baltimore Transit Co.* v. *Mezzanotti*, 227 Md. 8, 174 A. 2d 768 (Ct. App. 1961), as to liberal construction of the rules.

Massachusetts

No cases on point. Procedure continues to follow a practice act first adopted in 1852. 38 Mass. Gen. L. Ann. (1960).

Michigan

A complete procedural change designated the Revised Judicature Act. of 1961, and rules substantially similar to the

Federal rules to take effect Jan. 1, 1963. Mich. Gen. Ct. R. of 1963. *Hallett* v. *Michigan Consol. Gas Co.*, 298 Mich. 582, 299 N.W. 723 (1941), qualified protection of experts' reports; *Wilson* v. *Borchard*, 122 N.W. 2d 57 (Mich. 1963), qualified protection of work product.

Minnesota

Adopted rules virtually identical to the Federal rules, effective Jan. 1, 1952. 27A Minn. Stat. Ann., R. Civ. P. Dist. Ct. (1958). *In re Sandstrom's Estate*, 89 N.W. 2d 19 (Minn. 1958), production of documents denied for failure to show good cause; *Brown* v. *Saint Paul City Ry.*, 241 Minn. 15, 62 N.W. 2d 688 (1954), discovery of work product denied.

Mississippi

Limited discovery provisions, unlike the Federal rules. 2 Miss. Code 1942 §§ 1659, 1699 (1957). *Garraway* v. *Retail Credit Co.*, 141 So. 2d. 727 (Miss. 1962), qualified protection of experts' reports.

Missouri

Mo. Sup. Ct. adopted rules in 1959 similar to Federal rules effective Oct., 1960. 4 Mo. Rev. Stat., Sup. Ct. R. (1959). *State ex rel. State Hwy. Comm'n* v. *Jensen*, 362 S.W. 2d 568 (Sup. Ct. Mo. 1962), condemnation case protecting appraisers notes from discovery as work product; *State ex rel. St. Louis County Transit Co.* v. *Walsh*, 327 S.W. 2d 713 (Ct. App. Mo. 1959), photographs not privileged, per se, qualifies work product.

Montana

No cases on point. Mont. adopted the Federal rules almost verbatim, effective Feb. 9, 1961. Mont. Laws, 37th Sess., Ch. 13 (1961). As to extent of discovery under previous rules see, *State ex rel. Pitcher* v. *District Court*, 114 Mont. 128, 133 P. 2d 350 (1943).

Nebraska

No cases on point. As of 1951 Neb. has had discovery provisions similar to the Federal rules. Neb. Sess. Laws 1951, §§ 25.1267.01–25.1269 (1952).

Nevada

No cases on point. Nev. Sup. Ct. adopted rules similar to Federal rules, effective Jan. 1, 1953. 1 Nev. Rev. Stat., R. Civ. P. (1953).

New Hampshire

No provisions similar to Federal rules. 5 N.H. Rev. Stat. Ann., § 516 (1955). *McDuffey* v. *Boston & Maine R.R.*, 102 N.H. 179, 152 A. 2d 606 (Sup. Ct. 1959), permitting discovery of experts' reports; *Smith* v. *American Employer's Ins. Co.*, 102 N.H. 530, 163 A. 2d 564 (Sup. Ct. 1960), denying discovery of work product.

New Jersey

Rules substantially similar to the Federal rules became effective Sept. 15, 1948, and were revised in 1953. N.J. Practice, Part IV, Ch. 4 (1953). *Cermak* v. *Hertz Corp.*, 53 N.J. Super. 455, 147 A. 2d 800 (1958), discovery of experts' conclusions denied; *Kaplan* v. *Jones*, 77 N.J. Super. 31, 185 A. 2d 248 (Super. Ct. 1962), denying discovery of work product.

New Mexico

No cases on point. As of 1949 New Mexico has had rules similar to the Federal rules. 4 N.M. Stat. Ann. 1953, Ch. 21 (1954). *Salitan* v. *Carrillo*, 69 N.M. 476, 368 P. 2d 149 (1961), as to scope of discovery.

New York

In 1962 the N.Y. Civ. Prac. Laws and Rules were adopted to be effective Sep. 1, 1963. The rules do not expand the scope or methods of discovery nor make provision for pretrial conferences. N.Y. Laws, Civ. Prac. L. and R., Ch. 308 (1962). *Murphy* v. *City Products Corp.*, 188 N.Y.S. 2d 247, 17 Misc. 2d 1026 (Sup. Ct. Erie County 1959), denying discovery of experts' conclusions; *Hewitt* v. *State*, 216 N.Y.S. 2d 615, 27 Misc. 2d 930 (Ct. of Claims N.Y. 1960), condemnation case permitting discovery of experts' conclusions; *Pfaudler Permutit, Inc.* v. *Stanley Steel Service Corp.*, 212 N.Y.S. 2d 106, 28 Misc 2d 388 (Sup. Ct. Monroe County 1961), denying discovery of experts' conclusions; *Salzo* v. *Vi-She Bottling Corp.* 235 N.Y.S. 2d 585, 37 Misc. 2d 357 (Supp. Ct. Queens County 1962), qualified admission of experts' reports; *Cataldo* v. *Monroe County*, 238 N.Y.S. 2d 855, 38 Misc. 2d 768 (Supp. Ct. Monroe County 1963), qualified denial of insurance reports.

North Carolina

No cases on point. Limited discovery and deposition procedures. 1A N.C. Gen. Stat., recompiled 1953.

North Dakota

No cases on point. Rules similar to the Federal rules were adopted, effective July 1, 1957. 5 N.D. Cent. Code Ann., R. Civ. P. (1957).

Ohio

Procedure is under a legislative code first adopted in 1853. Ohio Rev. Code Ann., Ch. 2317 (1958) (1962 Supp.). *Neff* v. *Hall*, 170 N.E. 2d 77 (Ct. App. Ohio 1959). Condemnation case denying discovery of experts' reports; *Nomina* v. *Eggeman*, 188 N.E. 2d 440 (Ct. C.P. Ohio 1962), qualifying discovery of experts' conclusions; *In re Bates*, 167 Ohio St. 46, 146 N.E. 2d 306 (Sup. Ct. 1957), denying discovery of work product.

Oklahoma

No cases on point. Procedure is regulated by a code first adopted in 1870. Okla. Stat. Ann., Title 12, Ch. 10 (1960). *Application of Umbach*, 350 P. 2d 299 (Okla, 1960) Federal income tax returns held privileged.

Oregon

No cases on point. Code provisions regulate procedure and are much more limited than the Federal rules. 1 Ore. Rev. Stat., Ch. 45 (Ch. replaced 1961–1962). See 40 Ore. L. Rev. 94 (1960) as to work product.

Pennsylvania

Discovery procedures are not as liberal as comparable Federal provisions. Pa. R. of Ct. 1962. *Musulin* v. *Redevelopment Authority*, 25 D. & C. 2d 267 (Pa. 1961), condemnation cases denying discovery of appraisals and valuations; *Wright* v. *Philadelphia Transp. Co.*, 24 D. & C. 2d 334 (Pa. 1961), denying discovery of experts' reports and conclusions.

Rhode Island

No cases on point. Trend since 1956 to adopt procedure similar to Federal system, 10 R.I.B.J. 7 (Nov. 1961); 2 Gen. L. R.I., Title 9, Ch. 18 (1956). *De Courey* v. *American Emery Wheel Works*, 153 A. 2d 130 (R.I. 1959), as to court appointed experts.

South Carolina

No cases on point. Procedure still substantially the same as under the Field Code first adopted in 1870. 6 Code Laws S.C., Ch. 7 (1962). As to general provisions see, *Peagler* v. *Atlantic Coast Line R.R.*, 101 S.E. 2d 821 (S.C. 1958).

South Dakota

No cases on point. Rules have been adopted that are similar to Federal discovery provisions. 2 S.D. Code, Title 36, Ch. 36 (1960 Supp.).

Tennessee

No cases on point. Code provisions limited following some of the Federal provisions. 5 Tenn. Code Ann., Title 24 (1955) (Supp. 1962).

Texas

A detailed set of rules following the Federal provisions became effective Sept. 1, 1941. A series of amendments in 1957 substantially broadened the discovery procedure. Tex. R. Civ. P. (1955). *Harrell* v. *Atlantic Refining Co.*, 339 S.W. 2d 548 (Ct. Civ. App. Tex. 1960), discovery of work product denied.

Utah

Rules were adopted in 1950 which are very similar to the Federal rules. 9 Utah Code Ann., R. Civ. P. (1953). *Mower* v. *McCarthy*, 122 Utah 1, 245 P. 2d 224 (Sup. Ct. 1952), denying discovery of expert conclusions and work product.

Vermont

No cases on point. Discovery procedure similar to Federal procedure was adopted by statute in 1957 and substantially amended in 1959. 3 Vt. Stat. Ann., Title 12 (1961 Supp.).

Virginia

Sup. Ct. adopted a set of rules effective Feb. 1, 1950, with limited discovery procedure. 2 Va. Code, Title 8, R. Sup. Ct. App. (1950). *Cooper* v. *Norfolk Redevelopment & Housing Authority*, 197 Va. 653, 90 S.E. 2d 788 Sup. Ct. App. 1956), a condemnation case permitting discovery of experts' conclusions on agency principles.

Washington

No cases on point. Has adopted the Federal rules on discovery and pretrial conference. 0 Rev. Code Wash., R. Pleading, Prac. & P. (1960).

West Virginia

No cases on point. The W. Va. Sup. Ct. adopted rules similar to the Federal rules, effective July 1, 1960.

Wisconsin

Discovery statutes were amended in 1961 to harmonize with the liberal interpretation of the Federal provisions. 30, 38 Wis. Stat. Ann. (1958). *State ex rel. Reynolds* v. *Circuit Court*, 15 Wis. 2d 311, 112 N.W. 2d 686 (1961), condemnation case permitting discovery of appraisers' reports and opinions; *Walsh* v. *Northland Greyhound Lines*, 224 Wis. 281, 12 N.W. 2d 20 (1943), permitting discovery of experts' reports.

Wyoming

No cases on point. Wyo. Sup. Ct. adopted new rules similar to Federal rules in 1957. 2 Wyo. Stat. 1957, R. Civ. P. (1959). See, *Lake De Smet Reservoir Co.* v. *Kaufman*, 292 P. 2d 482 (Wyo. 1956) as to liberal interpretation of courts discretion in permitting discovery of books, documents, and papers.

APPENDIX V

Illustrative Statutory Provisions Regarding Expert Protection From Discovery

Absolute Protection

Absolute protection for the expert's report, the expert's conclusions, and the attorney's work product is illustrated by:

Smith-Hurd Ill. Ann. Stat. § 101.19-5 (Sup. Ct. R. 19-5)

§.101.19-5(1). *All matters* which are privileged against disclosure upon the trial are privileged against disclosure through any discovery procedure. Disclosure of memoranda, reports, or documents made by or for a party in preparation for trial or any privileged communications between any party or his agent and the attorney for the party shall not be required through any discovery procedure. (Emphasis added.)

Pa. R. of Ct. 1962

Rule 4011. Limitation of Scope of Discovery and Inspection. No discovery or inspection shall be permitted which: (a) is sought in bad faith; (b) causes unreasonable annoyance, embarrassment, expense, or apprehension to the deponent or any person or party; (c) relates to matter which is privileged or would require the disclosure of any secret process, development, or research; (d) would disclose the existence or location of reports, memoranda, statements, information, or other things made or secured by any person or party in anticipation of litigation or in preparation for trial or would obtain any such thing from a party or his insurer, or the attorney or agent of either of them, other than information as to the identity or whereabouts of witnesses; (e) would require the making of an unreasonable investigation by the deponent or any party or witness; (Items (a) through (e) were adopted November 20, 1950, effective June 1, 1951, amended April 12, 1954, effective July 1, 1954.) (f) would require a deponent, whether or not a party, to give an opinion as an expert witness, over his objection (Amend., March 1962, effective, April 1962.).

No Protection

No statutory protection for the expert's report or his conclusions and only qualified protection of the attorney's work product is provided by the following listed statutes:

Maryland Rules of Procedure, Rule 410.

§ 410(c). Writings Obtainable. Except as otherwise provided in rule 406 (similar to Federal rule 30(b) providing for protective orders), a party may by written interrogatory or by deposition require that an opposing party produce or submit for inspection:

(a) Report of Expert. A written report of an expert, whom the opposing party proposes to call as a witness, whether or not such report was obtained by the opposing party in anticipation of trial or in preparation for litigation. If such expert has not made a written report to the opposing party, such expert may be examined upon written questions or by oral depositions as to his findings and opinions.

§ 410(d). Writings Not Obtainable. Except as otherwise provided in rule 406, a party or deponent shall not be required to produce or submit for inspection:

(a) Object Prepared for Trial. A writing, statement, photograph or other object obtained or prepared in anticipation of litigation or in preparation for trial, except as provided in section C of this Rule, unless the court otherwise orders on the ground that a denial of production or inspection will result in an injustice or undue hardship.

(b) Reflecting Attorney's Conclusions. A writing which reflects an attorney's mental impressions, conclusions, opinions or legal theories. (In this case the attorney's work product appears to include only the results of the mental processes of the attorney.)

Qualified Protection

Qualifying protection of the expert's report upon the condition that it was prepared in preparation for trial and protecting any part is illustrated by the following listed statute.

Kentucky Revised Statutes, Rule 37.02.

Limitations on the Production of Writings. The deponent shall not be required and the court shall not order a deponent or party to produce or submit for inspection any writing obtained or prepared by the adverse party, his attorney, surety, indemnitor, or agent, *in anticipation of litigation or in preparation for trial unless satisfied that denial of production or inspection will result in an injustice or undue hardship;* nor shall the deponent be required or the court order a deponent or party to produce or submit for inspection *any part* of a writing which reflects an attorney's mental impressions, conclusions, opinions, or legal theories, or except as provided in rule 35, the conclusions of an expert. (Emphasis added.)

An Evaluation of Partial Taking of Property for Right-of-Way

BY THE
ECONOMIC RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by GEORGE V. BRODERICK, Economic Statistician, and FLOYD I. THIEL, Economist

Contrary to general public opinion, the partial taking of property for highway right-of-way usually has beneficial effects on the remaining property, according to the findings of the study of severance case records in the Bureau of Public Roads' files (bank). The information on the effects of the partial takings presented in this article is expected to be helpful to those concerned with the acquisition of right-of-way for highways—appraisers, negotiators, courts, and affected property owners.

As much background information as possible is needed to provide the basis for establishing a fair price for the purchase of right-of-way. To obtain this information, most State highway departments are conducting severance study programs and publishing the findings. Studies available and the effects of many partial takings reported to Public Roads by the States have been analyzed by the authors. From this analysis, it has been concluded that most property owners benefit from their encounter with highway departments.

Introduction

A MAJOR job facing builders of modern highways today is the equitable and timely acquisition of right-of-way. For several reasons, this task may be growing even more complex.[2] Controlled-access features of modern highways are placing more limits on abutters' rights. There is increasing competition for space, particularly in urban areas. And the problem is intensified by modern highway facilities needing wider rights-of-way.

Whether the task of acquiring right-of-way for highways is growing more difficult, there can be no doubt about the magnitude of this task. For the National System of Interstate and Defense Highways alone, a million and a half acres of land costing about $6.5 billion will be required by the completion date in 1972. Right-of-way acquisition in which the Federal Government participates is currently costing about $750 million a year—proposed State right-of-way programs for 1963, $685 million; for 1964, $757 million; for 1965, $870 million.

Severance Studies

To help assure that this money is being spent wisely, increasing use is being made of severance studies—case study analyses of the effect of taking part of a property for highway right-of-way. Such studies have been completed or are underway in 46 States, of which two-thirds have supplied information for inclusion in a central file or bank of cases that was established about 2 years ago (1961) in Public Roads. The States have supplied more than 1,200 case studies for this central file. The States have issued more than 1,500 individual case study reports, many of these are narrative reports or were made on State forms rather than on Public Roads forms.

Severance studies are intended to provide the information that will permit equitable payments to be made for property taken. By recording and analyzing the effect of partial taking of property for right-of-way in the past, severance studies make it possible to determine with more certainty the present and future effect of partial taking of properties for right-of-way. As more is learned about what happens to properties after part is taken for right-of-way, especially factors or characteristics that affect value, considerable savings in costs are expected to be realized. But severance studies obviously are not intended simply to reduce costs of right-of-way acquisition. Inadequate payments for right-of-way are fully as alarming to conscientious highway builders as excessive payments.

Summary

The findings presented in this article are tentative; they are only typical of the cases analyzed and are not representative of *all* cases. The tentative findings may change when more cases become available for analysis.

The high cost of right-of-way, more than a billion dollars a year, and the need at the same time to provide just compensation when acquiring right-of-way, provide a strong momentum for examining past experience to learn what general truths might be useful for right-of-way acquisition in the future. By organizing and making available in usable form the experience gained in highway acquisition, severance studies offer a way of correcting certain overpayments as well as the relatively few underpayments for highway right-of-way. Many State highway departments are now enjoying this benefit as the result of their own severance studies. In addition to this use of severance studies, which must be regarded as their primary purpose, findings from analyzing a collection of cases can be expected to provide some guidelines for right-of-way acquisition in the future. Although information in the Public Roads' bank of cases does not now permit formulas to be developed to predict the experience owners will have with their remainder parcels, some tentative observations can be made, as follow:

(1) The recovery rate for cases in the Public Roads' bank tends to be more than 100 percent, the median is 138 percent.

(2) Certain characteristics tend to be associated with a higher-than-average recovery rate. These include: nearness to an interchange, a sale after some period of time (e.g., more than a year) after the taking, a vacant rather than a residential land use before the acquisition, and full visibility of the highway from the remainder.

(3) When the simultaneous effect of several factors acting in combination was analyzed by multiple regression, the most influential factors were: a change in land use, time elapsing from acquisition to sale, travel distance to new highway, type of remainder, and nearness to an interchange.

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[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964, under the title of *Highway Severance Damage Studies—Some General Findings.*

[2] For a brief discussion of the growing complexity of right-of-way acquisition, see *An Editorial,* Right-of-Way, vol. 10, No. 5, October 1963, p. 5.

(4) The owner is being made whole in four out of five cases.

(5) Property owners who lost generally had lost very little. Gains ranged from small amounts to fantastically large gains.

(6) Owners of residential properties are more likely to experience losses than owners of land in other uses. Gains are often associated with vacant remainders.

(7) Damage payments made to owners of vacant parcels often are unrealistically high. Experience suggests that high damage payments for vacant properties partially taken should receive close scrutiny in the future.

Benefits of Severance Studies

Many of the benefits to be derived from severance studies are already being realized. These studies help assure the proper spending of tax money for right-of-way purposes; they make available information relevant to the takings. This information is needed by appraisers and negotiators, the courts, and affected property owners, if the State's purpose to buy right-of-way property at a fair price is to be accomplished.

Analysis to supply experience in similar situations—the purpose of individual severance studies—is the traditional approach employing data for comparable situations, which has been used successfully by appraisers. Ordinarily, the best sources for comparable information in taking situations are studies completed within the State, and most States do rely on data obtained from such cases. For unusual situations—takings involving special purpose properties—the Public Roads' bank can be searched for comparable takings. The procedure for requesting a search is described on page 93 of the *Manual for Highway Severance Damage Studies*, and the type of data that can be obtained is shown in table 1.

A fairly common result of severance investigations shows that (1) after a partial taking for right-of-way, the adverse effect on remaining land parcels is often much less drastic than feared or (2) the remainder parcel receives a significant benefit. Thus, these studies can be useful in keeping affected individuals and the general public informed.

Collection of cases

Collecting severance cases offers opportunities for analyzing these cases. Although the data reflected in the bank of cases cannot be considered typical for all highway takings, the data that can be assembled permit some interesting and perhaps valuable comparisons. Although there are now about 1,250 cases in the bank, cases are not usable for analysis until they have been edited and checked. The number of usable cases for different comparisons varies. For example, more than 900 cases can be used to compare the per acre value at the time of the highway taking with the per acre value of the remainder that is sold, and the 650 cases in the bank for which the entire remainder has been sold provide a good indication of the extent to which the *owner was made whole* or, in a very general way, whether just compensation was provided.

Recovery Rate Experience

The recovery rate for a highway-severed parcel is obtained by dividing the value per acre (or per square foot) of part or all of the remainder that has been sold by its value at the time of the taking. A recovery rate of more than 100 percent means that the remainder has increased in value. As the recovery rate can be determined when any part of a remainder is sold, this type of comparison ordinarily can be made for a case as soon as any portion of the remainder has been sold.

Because of the extremely high recovery rates for some remainder parcels, simple arithmetic averages may not be a satisfactory summary measure of the typical recovery rate for severed parcels in the bank at the present time (1963). Median values provide another way of summarizing the overall recovery rate. As a median is a middle value with half of the cases above and half of them below, those remainder parcels having extremely high recovery rates do not have such a noticeable effect on median values as on average values. The median recovery rate for cases in the bank at the end of 1963 was 138 percent. About 75 percent of all cases showed a recovery rate of more than 100 percent, as shown in figure 1. Some 7 percent of the cases showed a recovery rate of more than 1,000 percent, and 25 percent of the cases showed a recovery rate of less than 100 percent.

In addition to considering recovery rates reported for all cases in the bank, rates have been compared according to (1) time of the sale, (2) land use before the taking, (3) type of highway involved, (4) visibility from remainder parcel, and (5) location of the parcel in relation to an interchange.

Table 1.—Comparison of principal characteristics of property and comparable property

Item	Subject parcel	Comparable sale
Land use before	School	Elementary school.
Land use after (expected)	(School)	Retail.[1]
Size before	10 acres	11 acres.
Size after	8 acres	8 acres.
Highway characteristics	Interstate	Interstate.
Value before	$70,000	$69,000.
Value of portion acquired	$20,000	$18,000.
Estimated benefit (+) or damage (−).		−$15,000.
Estimated remainder value.		$36,000.
Sales price of remainder		$89,000.
Effect of taking		+$38,000.

[1] Although the elementary school was expected to continue as a school, the use changed to retail soon after the taking. In this case, which is recorded in the Bureau's bank, dollar amounts have been rounded to the nearest hundred.

Time of sale

Whether the time at which a remainder parcel sells has any effect on the recovery rate has been the subject of considerable speculation. The effect of time is of interest because it has a bearing on the validity of the comparison between before values and after values. If a sale occurs soon enough after the taking, the highway effect is revealed by simply comparing the before value with the value shown by the sale.

The effect of time on recovery rates of cases in the bank is very noticeable. Remainder parcels that are sold a year or more after the time of the taking tend to have higher recovery rates. As can be seen in figure 2, parcels that were sold within a year's time had a lower rate of recovery. A third of the parcels that were sold within the first year had a recovery rate of less than 100 percent. For parcels sold more than 3 years after the highway taking, only 12 percent had a recovery rate of less than 100 percent. Nearly 60 percent of the land parcels that were sold more than 3 years

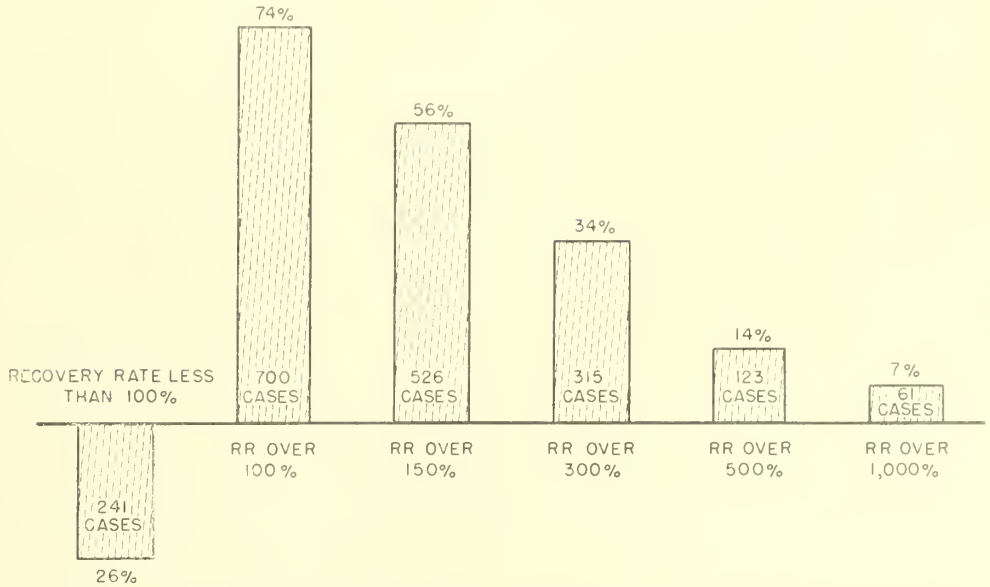

Figure 1.—Land value recovery rates—overall recovery rates by number and percentage of cases.

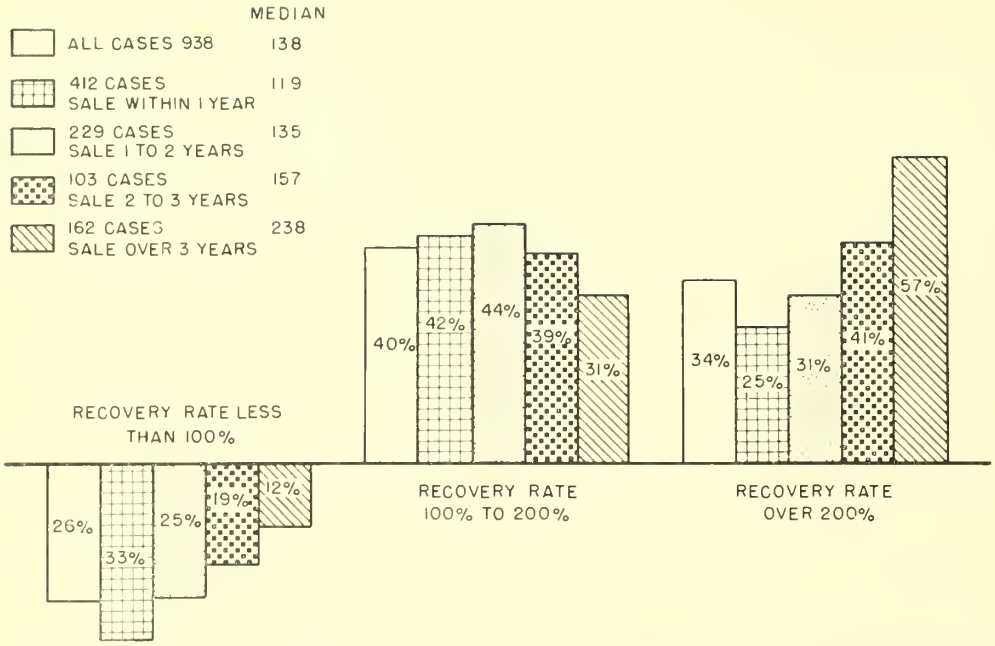

Figure 2.—Land value recovery rates by time from acquisition to sale—unadjusted for general land value changes.

after the highway taking had a recovery rate of more than 200 percent, and about 15 percent had a recovery rate of 1,000 percent or more. In contrast, only about 25 percent of the land parcels that were sold within the year of the taking had a recovery rate of more than 200 percent; 4 percent had a recovery rate of 1,000 percent or more.

The median recovery rates for parcels sold at different lengths of time after the highway taking emphasize the effect of time. The median recovery rate for property sold within 1 year was 119 percent; for property sold between 1 and 2 years after the taking, 135 percent; for property sold between 2 and 3 years after the taking, 157 percent; and for property sold more than 3 years after the taking, 238 percent. This is shown in figure 2. These median recovery rates adjusted for general land value increases (an average annual increase of 7 percent was used) are still spectacular: 115 percent, 121 percent, 129 percent, and 155 percent, respectively. Thus, it appears that land values of affected parcels tend to appreciate in value considerably faster than is true for land values generally. Eventually, when enough cases are available for analysis, it may be possible to limit the comparison to cases where the remainder is sold very soon after the acquisition. Such a comparison would generally exclude the general land value increase occurring over a period of time and leave only the highway effect. With such a simple before and after comparison, the effect of characteristics other than time (e.g., type of land use, type of highway system) should become more easily distinguished.

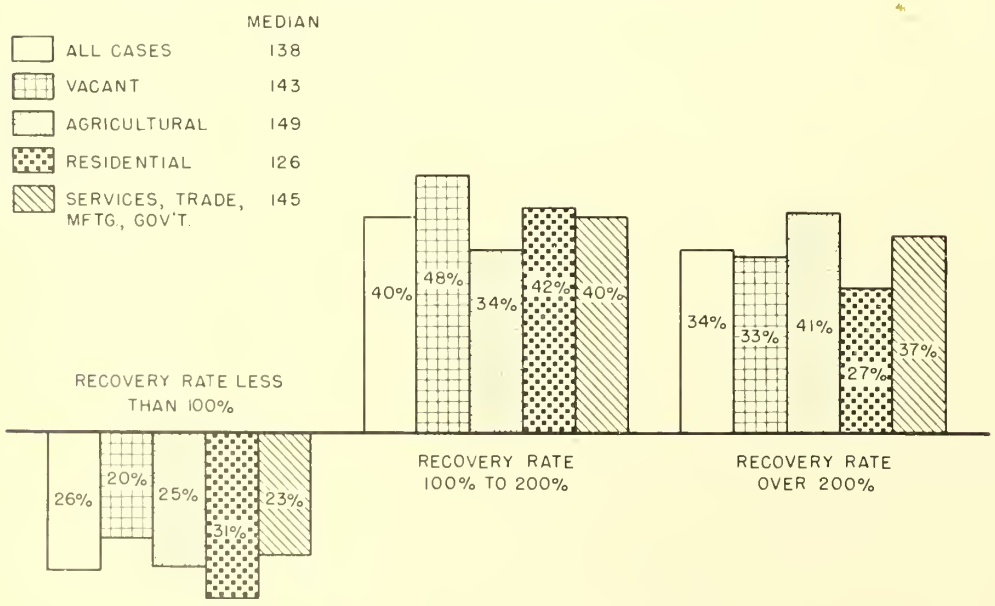

Figure 3.—Land value recovery rates by land use at the time of acquisition.

Land use

Another characteristic that appears to have an effect on the recovery rate is the use that the land was put to at the time of the highway taking (fig. 3). The median recovery rate for residential property, for example, is about 126 percent compared with a median recovery rate for all cases of 138 percent. The other land uses—vacant, agricultural, and a combination of services, trade, manufacturing, and government—had recovery rates of 143 percent, 149 percent, and 145 percent, respectively. The relatively poorer recovery rates for residential property is highlighted by the bar charts in figure 3. For example, only 27 percent of the residential property remainders had a recovery rate of 200 percent or more, and 31 percent of the residential property had a recovery rate of less than 100 percent.

Type of highway system

Another comparison, by type of highway system, shows some differences that may be attributable to whether the remainder parcel was located on an Interstate highway, a Federal-aid primary highway, or a Federal-aid secondary road. The median recovery rate for remainder parcels along Interstate routes is about 140 percent, slightly higher than the median recovery rate (138 percent) for all cases in the bank. The recovery rate is about 132 percent along Federal-aid primary highways, and about 135 percent along Federal-aid secondary roads.

In addition to somewhat higher recovery rates, for remainder parcels along the Interstate System more large gains and more losses have been experienced than for parcels along other highway systems. As shown in figure 4, about 35 percent of the remainder parcels located along Interstate Highway Systems have had recovery rates of more than 200 percent. This is a slightly larger portion of parcels than the remainder parcels located along Federal-aid primary systems and Federal-aid secondary systems. At the same time, about 30 percent of the remainder parcels located along the Interstate System have had recovery rates of less than 100 percent, compared with about 24 percent of the remainders along the Federal-aid primary system and 26 percent of the remainders along Federal-aid secondary systems, which had recovery rates of less than 100 percent. Whether the recovery rates along Interstate routes will continue at the same level when more cases are available to analyze is not clear. Perhaps the overall recovery rates for remainder parcels along Interstate routes will be more spectacular than for remainder parcels located along other types of highway systems.

The higher-than-normal recovery rates along Interstate routes might be expected, but it may be that recovery rates for many parcels located along the Interstate route will be lower than for parcels located on other types of highway systems because of the lack of direct access to the Interstate System. However, the contrast between Interstate and non-Interstate recovery rates is sharper

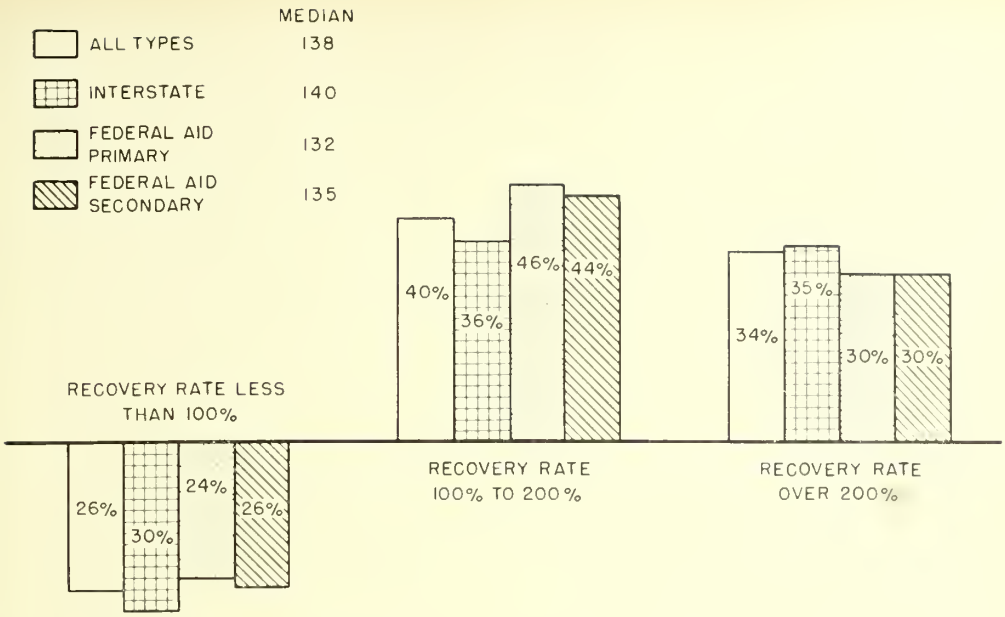

Figure 1.—Land value recovery rates by type of highway system.

at the upper range of recovery rates than it is at the lower end. Thus, the recovery rates along the Interstate System are distinguished from those for other highways primarily by the high recovery rates; when recovery rates are low along the Interstate System, the rates are only slightly different from those along other types of roads.

Visibility from the remainder

The States that are sending severance cases to the Public Roads' bank are providing information as to whether the highway is visible from the remainder parcel. Most of the time full visibility means that the property is also visible from the highway. Tentative analysis of the recovery rates by visibility shows some interesting differences, though it is not now possible to tell just how significant these differences are. The median recovery rate for parcels from which the highway is fully visible, for example, is 145 percent, compared with a recovery rate of 133 percent for parcels from which the highway was partially visible, and 117 percent for parcels from which the highway could not be seen. This is shown in figure 5, along with the median recovery rate for all cases—138 percent. Also, 37 percent of those remainder parcels from which the highway could be seen fully had a recovery rate of more than 200 percent, but only about 21 percent of the remainder parcels from which the highway could not be seen had such a high recovery rate.

As noted earlier, the significance of the recovery rates cannot be fully discerned at this time; however, the claims that are often made about the undesirable appearances and effects of modern highway improvements have seldom been substantiated. Apparently the market does not discount the value of property from which the highway can be seen. On the contrary, property from which the highway can be seen appears to fare better in the market place than property from which the highway is not visible.

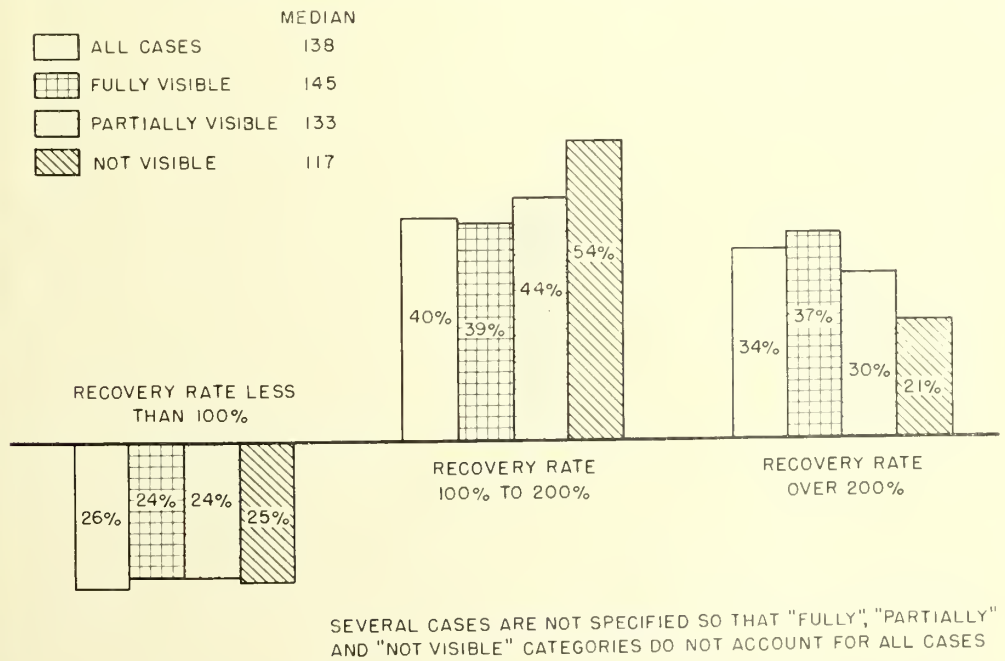

Figure 5.—Land value recovery rates by visibility of highway from remainder.

Interchange effects

What happens around interchanges is depicted in figure 6. Approximately one-fourth of the 900 plus cases used in this analysis were located within a half mile of an interchange, a distance often used to distinguish between interchange and noninterchange areas. The recovery rate of parcels located within a half mile of an interchange is generally better than the recovery rate for parcels located farther away from an interchange. For example, the median recovery rate for parcels located near interchanges was about 164 percent compared with 131 percent for parcels located away from the interchange. Also, more of the interchange properties had high recovery rates than was true for parcels located away from the interchange. Nearly half of the parcels located within a half mile of an interchange had recovery rates of more than 200 percent.

Multiple Regression Analysis

In analysis of the recovery rates of highway-severed remainders, an examination of the influence of several factors taken one at a time generally has been relied upon. In the investigation described here, a start has been made to determine the simultaneous effect on the recovery rate of several factors, acting in combination, and to measure the relative strength of each of the factors. For this analysis, the technique of multiple regression has been used.

When the simultaneous effect on the recovery rate of several factors acting in combination was studied, the most influential factors were (1) change in land use, (2) time elapsing from acquisition to sale, (3) travel distance to the new highway, (4) type of remainder (landlocked, isolated, or separated), and (5) nearness to interchange. For one of the groups of cases studied, a coefficient of multiple correlation of 0.86 was obtained, indicating that 73 percent of the total variation in the recovery rate was explained by the combined effect of the several independent factors used in the analysis. Additional and more refined analysis of this kind is planned for the future.

Are Public Roads' Cases Typical

As many of the States supplying information about remainder parcels do not report on all remainder parcels in the State or on a representative sample of them, some question may exist as to whether the cases in the Public Roads' bank are typical of partial taking in general. Although there appears to be no definitive test that would answer this question, a check can be made to compare the findings from the bank as a whole with the

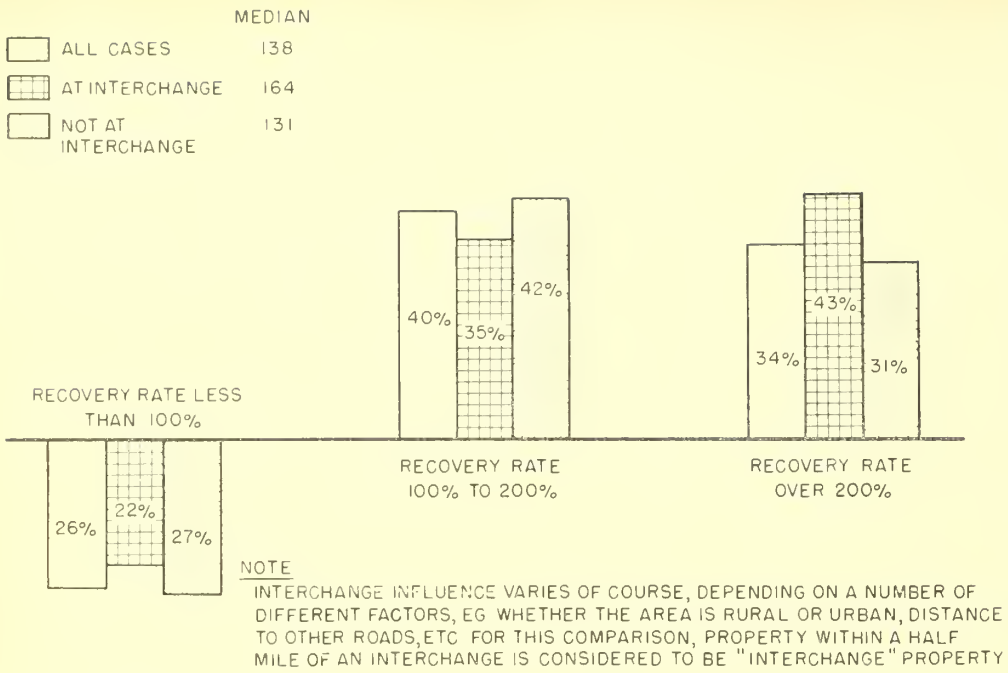
MEDIAN
ALL CASES 138
AT INTERCHANGE 164
NOT AT INTERCHANGE 131
RECOVERY RATE LESS THAN 100%
RECOVERY RATE 100% TO 200%
RECOVERY RATE OVER 200%
26%
22%
27%
40%
35%
42%
34%
43%
31%
NOTE
INTERCHANGE INFLUENCE VARIES OF COURSE, DEPENDING ON A NUMBER OF DIFFERENT FACTORS, EG WHETHER THE AREA IS RURAL OR URBAN, DISTANCE TO OTHER ROADS, ETC FOR THIS COMPARISON, PROPERTY WITHIN A HALF MILE OF AN INTERCHANGE IS CONSIDERED TO BE "INTERCHANGE" PROPERTY

Figure 6.—Overall land value recovery rates by nearness to interchange.

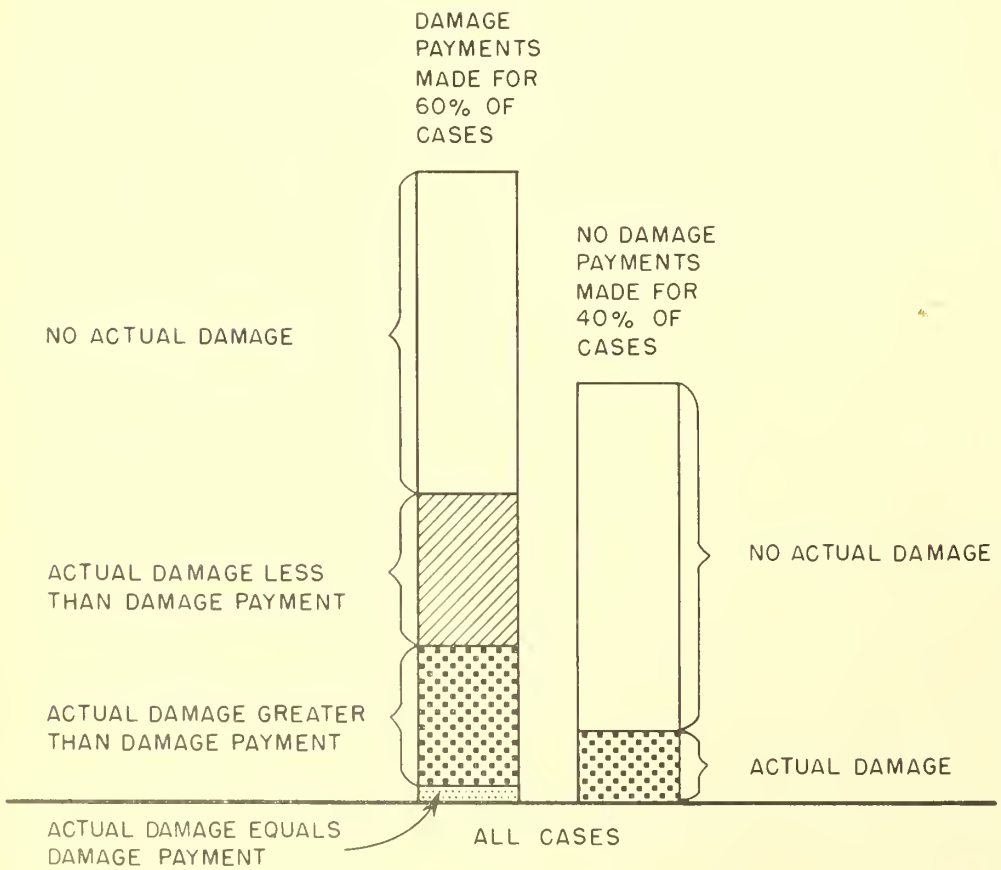
DAMAGE PAYMENTS MADE FOR 60% OF CASES
NO DAMAGE PAYMENTS MADE FOR 40% OF CASES
NO ACTUAL DAMAGE
ACTUAL DAMAGE LESS THAN DAMAGE PAYMENT
ACTUAL DAMAGE GREATER THAN DAMAGE PAYMENT
NO ACTUAL DAMAGE
ACTUAL DAMAGE
ACTUAL DAMAGE EQUALS DAMAGE PAYMENT
ALL CASES

Figure 7.—Comparison of damage payments with actual damages.

findings from a State that is supplying information about all remainder parcels that have been sold. This has been done. Findings for all cases in the bank have been compared with those of the approximately 400 California cases, which are in the bank.

The findings for all cases compare fairly closely with those based solely on California cases. For example, the median recovery rate reported for California cases is about 142 percent compared with 138 percent for the entire bank. The comparison was made between findings from California cases and all cases, rather than between findings from California cases and all non-California cases, primarily for convenience. It seems fairly obvious that the differences between data in California and non-California cases would be slightly greater than those between California cases and all cases. Properties located within a half mile of an interchange had a median recovery rate in California of 166 percent, compared with a recovery rate of 164 percent for the bank as a whole. The percentage of cases reported by California for which the property was located within a half mile of an interchange—about 25 percent—agrees generally with the percentage of all bank cases in which the property was near an interchange —about 29 percent. Thus, it appears that there are similarities in the effects reflected by the California cases and the total effects reflected by those in the bank, except that the recovery rates in California are slightly higher than the recovery rates in other States.

Extent to which the Owner is Made Whole

Whether the owner is made whole can be determined by comparing before and after property values. When a State takes part of an owner's property for highway right-of-way, and then after a period of time the owner sells the entire remainder, it can be said that all the results are in for that owner and for that property. The appraised value of the entire tract before the taking is known; the payments made by the State to the owner for the property taken, as well as for any expected damages to the remainder, are known; and the sale price of the entire remaider is known. It is then possible to determine whether the owner was damaged or benefited, and the extent of the damage or benefit can be determined.

A before and after examination of the 650 cases in the Public Roads' bank in which the entire remainder was sold reveals the extent to which owners of property partially taken for highway right-of-way were made whole—that is, whether affected property owners were placed in as good a financial position as they would have been had their property not been taken. To measure the effects of the partial taking of property for each of the 650 cases selected, the value of the entire property (including improvements) before the taking was compared with the total amount the owner received from the property; that is, for the property taken, for damages to the remainder, and from the sale of the entire remainder.

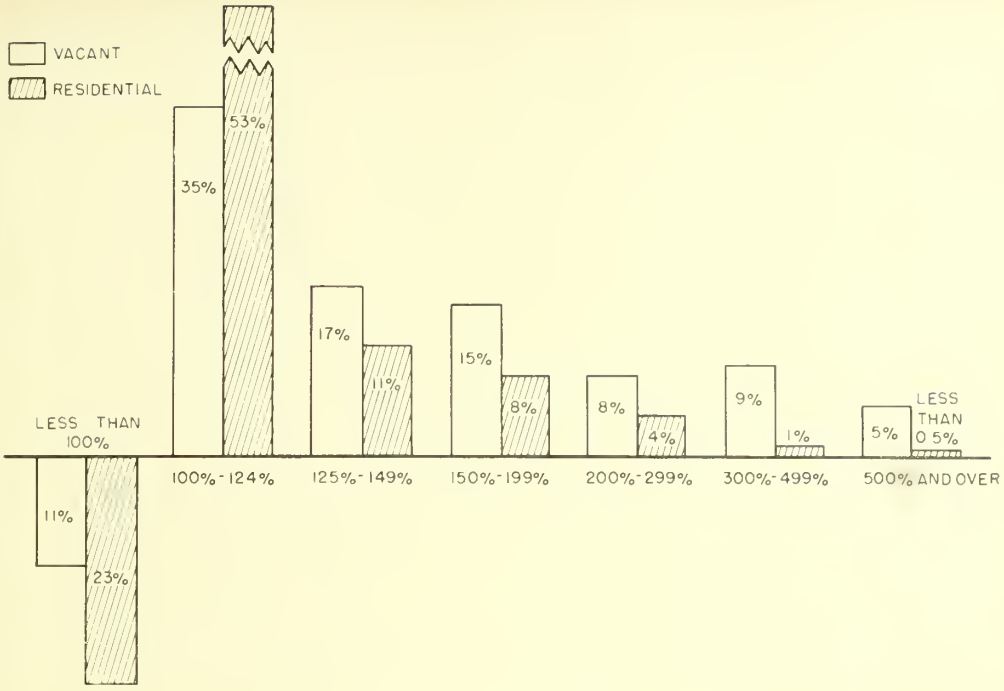

Figure 8.—Percentage distribution of value received as percent of value by before land use, vacant and residential.

APPRAISED BEFORE VALUE ENTIRE TRACT (INCLUDING IMPROVEMENTS)	$14,977,800
PAYMENT FOR PROPERTY TAKEN (EXCLUDING DAMAGES)	4,011,600
PAYMENT FOR DAMAGE TO REMAINDER	1,563,600
TOTAL PAYMENTS BY STATE	5,575,200
SALE PRICE OF ENTIRE REMAINDER	15,311,500
TOTAL RECEIPTS OF PROPERTY OWNERS	$20,886,700

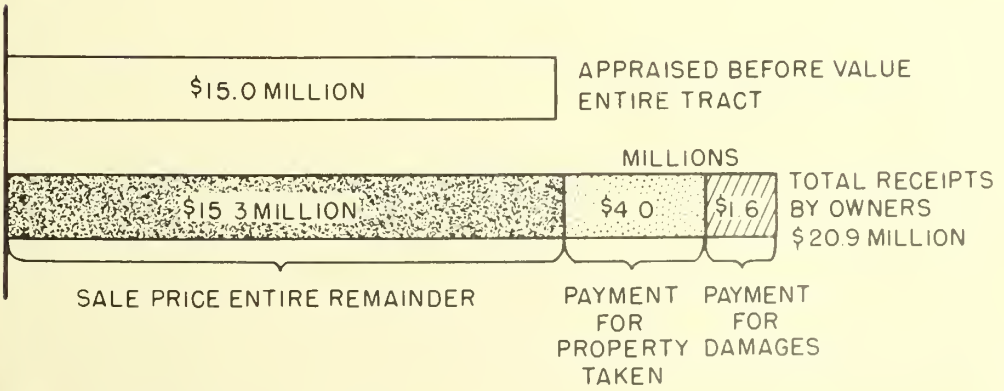

Figure 9.—Comparison of total appraised value before with total payments received by owners.

For the cases analyzed, four out of five property owners received either adequate compensation or more. The remaining 20 percent of the property owners ended up with less money after the highway taking than they had in property before the highway improvement. The median value that the entire group of 650 property owners received was 112 percent of the before value of their property.

Damages—Estimated and Actual

For the 650 cases analyzed, damage payments were made to the owners of 60 percent of these properties; the remaining 40 percent received no payments. Examination of the experience of owners receiving damage payments revealed that half of the recipients actually sustained no damage at all, and one-fourth of the recipients of damage payments suffered less actual damage than they were paid for. A fifth of all recipients of damage payments received less in damage payments than the cost of damage they actually sustained. Of the owners who received no damage payments, more than four-fifths experienced no actual damage and the remaining fifth suffered actual damage. Thus, for both groups, about one owner in five suffered a loss as the result of an under payment of damages or the nonpayment of damages. Highway officials are, of course, just as concerned about property owners receiving inadequate compensation as they are about apparent overpayment of damages: The goal is to make the owner whole. A comparison of these findings is presented in figure 7.

Damage Payments Compared to Total Payments

It is of interest to compare the proportion of total State payments accounted for by damage payments for selected categories of partial taking cases with total combined payments for all cases. Using total combined payment figures, damage payments accounted for 28 percent of total payments made by the States for right-of-way acquisition. However, for vacant land nearly half the cost of acquisition was accounted for by damage payments.

Why damage payments are so high for vacant land remainders in contrast to the higher-than-average recovery rates for vacant property is somewhat perplexing. The result is that owners of vacant land have been treated better than owners of other types of property. For example, owners of vacant land had receipts amounting to 129 percent of the before value of their property compared with 107 percent for owners of residential properties. This contrast in value received as a percent of before value as between vacant parcels and residential parcels is highlighted by figure 8. Owners of vacant parcels had fewer losses than residential property owners (11 percent versus 23 percent). And, a much higher proportion of owners of residential than of vacant properties experienced relatively small gains over the before value. It

is clear that owners of vacant properties generally fared better than residential land owners.

At least a partial explanation of the more favorable after-taking experience of owners of vacant land is given by still another finding from the bank. A comparison of the uses of remainder parcels at the time they were sold, with their uses at the time of the taking, revealed that nearly a third of those parcels vacant at the time of taking had shifted to higher uses by the time the parcel was sold. By contrast, less than a tenth of residential parcels had shifted to higher uses by the time they were sold. In view of these findings, it appears that the acquisition of vacant land offers a good chance for improvement in the pursuit of the goal of making the owner whole.

Total Values Compared

The experience of individual owners following the partial taking of their property for highway right-of-way has been examined and presented in the form of frequency distributions, percentage distributions, and medians. Now, the total experience of affected owners, obtained from examination of the entire bank of partial taking cases in which the entire remainder was sold, and the experience of different groupings of these owners is discussed. The total of the appraised before values of the properties of the 647 owners was $15 million. The owners of these properties were paid a total of $4 million for property taken (exclusive of damage payments) and $1.6 million in damage payments. Finally, these owners sold their remaining property for a total of $15.3 million (fig. 9).

If these figures are adjusted for the general increases occurring in land values, the expected total market value is $10.2 million. A comparison of this very rough estimate of the expected total market value of the remainders at the time of sale with the actual total sale price gives a rough idea of the extent of land value increases and/or overpayments for damages. Remainders that might have been expected to sell for $10.2 million were sold for $15.3 million. This is an oversimplification because some State laws do not permit the use of benefits to offset the cost of taking or against damages to the remainder. Thus, even after considering a general increase in land values, the total receipts of affected owners were considerably higher than the total before value of their property.

This finding of large total receipts, of course, should in no way be understood to imply that severance damages should not be paid. Two out of five affected owners did actually suffer damage. One of these received either insufficient payments or no damage payments. In fact, the only purpose served by this kind of total analysis is to indicate the outside theoretical limits of the improvement that might be made in the awarding of damages to owners of highway-severed properties. However, it appears that very careful consideration should be given to the offsetting of benefits against damage payments where appropriate, and to the offsetting of benefits against payments for property taken where appropriate and where State law permits.

NEW PUBLICATIONS

Highway Progress, 1963

Annual Report of the Bureau of Public Roads, Fiscal Year 1963

A review of the accomplishments of the Bureau of Public Roads, U.S. Department of Commerce, during the fiscal year 1963, particularly those related to the Federal-aid highway program, is presented in the annual report, *Highway Progress, 1963.* The 109-page illustrated publication contains a descriptive account of the progress made during fiscal year 1963 on construction of the National System of Interstate and Defense Highways, and information on the improvement of primary highways, secondary roads, and urban arterials under the regular Federal-aid program.

Also described in the annual report is the highway construction work undertaken directly by the Bureau of Public Roads in national forests and parks and on other Federal lands, as well as Public Roads activities in providing technical assistance to foreign countries to further their programs of highway development.

Reported on at length are the activities and accomplishments of Public Roads in management, highway planning and design, urban transportation planning, safety, and in the extensive and varied research and development program. Statistical information on the progress and activities of the Federal-aid program during the fiscal year 1963 is presented in 19 statistical tables included as an appendix to the report.

Highway Progress, 1963, may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for 35 cents a copy.

U.S. GOVERNMENT PRINTING OFFICE: 1964

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–32 are available upon request addressed to Bureau of Public Roads, Washington, D.C., 20235.*

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Annual Reports of the Bureau of Public Roads:

1951, 35 cents. 1955, 25 cents. 1958, 30 cents. 1959, 40 cents. 1960, 35 cents. 1962, 35 cents. 1963, 35 cents. (Other years are now out of print.)

REPORTS TO CONGRESS

Factual Discussion of Motortruck Operation, Regulation and Taxation (1951). 30 cents.

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:

First Progress Report, House Document No. 106 (1957). 35 cents.

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

Final Report, Part VI: Economic and Social Effects of Highway Improvement, House Document No. 72 (1961). 25 cents.

The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

U.S. HIGHWAY MAP

Map of U.S. showing routes of National System of Interstate and Defense Highways, Federal-Aid Primary Highway System, and U.S. Numbered Highway System. Scale 1 inch equals 80 miles. 25 cents.

PUBLICATIONS

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

Calibrating and Testing a Gravity Model With a Small Computer (1964). $2.50.

Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

PUBLICATIONS—Continued

Federal Laws, Regulations, and Other Material Relating to Highways (1960). $1.00.

Financing of Highways by Counties and Local Rural Governments: 1942–51 (1955). 75 cents.

Highway Bond Calculations (1936). 10 cents.

Highway Bond Financing . . . An Analysis, 1950–1962. 35 cents.

Highway Capacity Manual (1950). $1.00.

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

Highway Statistics (published annually since 1945):

1955, $1.00. 1956, $1.00. 1957, $1.25. 1958, $1.00. 1959, $1.00. 1960, $1.25. 1961, $1.00. 1962 $1.00.

Highway Statistics, Summary to 1955. $1.00.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Hydraulics of Bridge Waterways (1960). 40 cents.

Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). 40 cents. Appendix, 70 cents.

Interstate System Route Log and Finder List. 10 cents.

Landslide Investigations (1961). 30 cents.

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Moyie River Bridge, Moyie Springs, Idaho (April 1964).

Completion of construction of the Moyie River Bridge, part of a Federal-Aid Forest Highway Project, will eliminate hazardous driving conditions in this area by a relocation of U.S. 2. The old route of U.S. 2 is shown in foreground and along the side of the canyon on the right. Note the safety nets in use during construction.

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IN THIS ISSUE

Selecting the Best Scale for In-Motion Weighing, *by R. E. Puckett* 45

Effect of Moisture on Bituminous Pavements in Rocky Mountain Areas, *by W. L. Eager* 48

Stability of Bituminous Pavements Related to Aggregate Characteristics, *by R. E. Olsen* . . . 54

Map of *The National System of Interstate and Defense Highways*, March 31, 1964 56

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Selecting the Best Scale for In-Motion Weighing

by **RUSSELL E. PUCKETT, P.E.**, Assistant Professor of Electrical Engineering, College of Engineering, Engineering Experiment Station, University of Kentucky

Factors that might affect the choice of the type of scale to be used in a dynamic weighing system have been considered in the study of the characteristics of two dissimilar scale platforms reported in this article. Each of the two types of scales has been deemed adequate for the purpose for which it was designed. The determining factor in the selection of the type of scale is the desired form of the ultimate recorded output from the in-motion weighing system, in which the platform will be the weight sensor. One of the scales yields an output that is a discrete value of a rapidly varying applied weight; the output of the other occurs over a relatively long period of time so that its output varies according to the variation in the load on the platform.

Introduction

THE PROBLEM of selecting the best type scale for in-motion weighing has been the basis for an active research program at the University of Kentucky during the past 4 years. The research project is sponsored jointly by the U.S. Bureau of Public Roads and the Kentucky Department of Highways. The primary objective of the research has been to determine the best mechanical configuration of a weighing platform to be used in a system to measure the axle weights of moving vehicles. This project was conducted as a part of the overall investigation of problems related to the development of a dependable, automatic, electronic weighing system and traffic data collection system.

A thorough investigation of the operating characteristics of two dissimilar configurations of scale platforms was made in this testing program, which was initiated by the research group and devised to satisfy the objective of the project. An experimental test site was set up on the approach to a static-weighing station. This location was chosen because it would permit rapid correlation of dynamic weight and static weight of each measured axle of passing vehicles. The primary objective was to assure that each of the platforms would yield an output that was a direct analog of any load applied to it. Lesser objectives were studies of preloading methods and their effects on the outputs of the scales. Some of the findings of the research group have been reported previously (*1*, *2*).[1]

Conclusions

On the basis of the research reported here, the conclusion has been reached that the choice of the type of scale for an electronic in-motion weighing system should be determined by the ultimate form desired for the recorded weight measurements and the end-use to be made of these measurements. From the comparison of the characteristics and capabilities of the two types of platform scales, it has been concluded that the broken bridge type of scale is the better choice when measurements of discrete values of weight are necessary, as in the classification of loads into weight groups. But the rigid platform type of scale was concluded to be the better choice when in-motion measurements are to be made and recorded and no manual manipulation is to be made of the recordings, as in an automatic data collection system.

The Scale Platforms

Two scale platforms were installed at the test site on Interstate 64, near Shelbyville, Ky., on the approach lane to a static-weighing station. One of the scales was a commercial design of the Taller-Cooper Co., a 4-point supported platform; the other scale was an experimental design based on the German broken bridge type of scale (*3*). The mechanical arrangements of the two platforms are sketched in figures 1 and 2. For the purposes of the testing program, pretensioning of the platforms on their supporting load cells, called *preload*, was accomplished by steel rods and turnbuckles for the Taller-Cooper scale and by heavy coil springs for the broken bridge scale.

Each scale platform uses strain-gage load cells as the weight-sensing transducers in the measuring system. The Taller-Cooper platform uses four 20,000-pound load cells, and the broken-bridge scale uses two 50,000-pound load cells. The Taller-Cooper platform is supported on the four cells, one at each corner of the platform; the broken-bridge platform is hinged at its outer edges and supported in the center by the two load cells. Each load cell is electrically driven at its input by transformer coupling from a 600-c.p.s. master

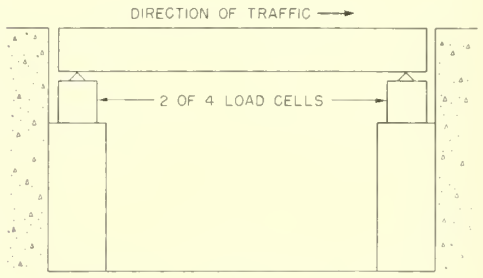

Figure 1.—Taller-Cooper scale.

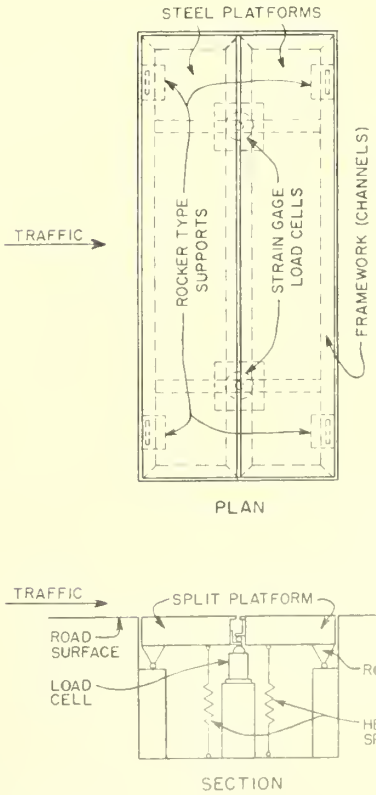

Figure 2.—Broken bridge scale.

[1] References indicated by italic numbers in parentheses are listed on p. 47.

oscillator power amplifier (MOPA). The input signal is common to all load cells in both platforms. Transformer coupling of the input driving signals precludes interaction of the load cells when a load is applied. The MOPA unit was installed at the static scale house with the analog recording equipment. About 600 feet of interconnecting cable was used between the pits containing the load cells and the scale house. The output signals of all load cells in each of the platform pits were electronically added together so that the total output of each scale would be directly proportional to any vertical load applied to it. In this way, the output from the scale would be a direct analog of the axle weight of a passing vehicle.

The output signals from both scales were received in the static scale house and, after necessary signal conditioning had occurred, the signals were recorded on a 2-channel Sanborn paper chart recorder. When the chart of the recorder had been calibrated, the recorded traces could be analyzed to determine several important facts concerning the passing vehicle, including its speed, the weight of each axle, and its axle spacing. The theoretical shapes of the expected output signals from each scale are shown in figure 3, and some actual recorded traces from the two scales are shown in figure 4.

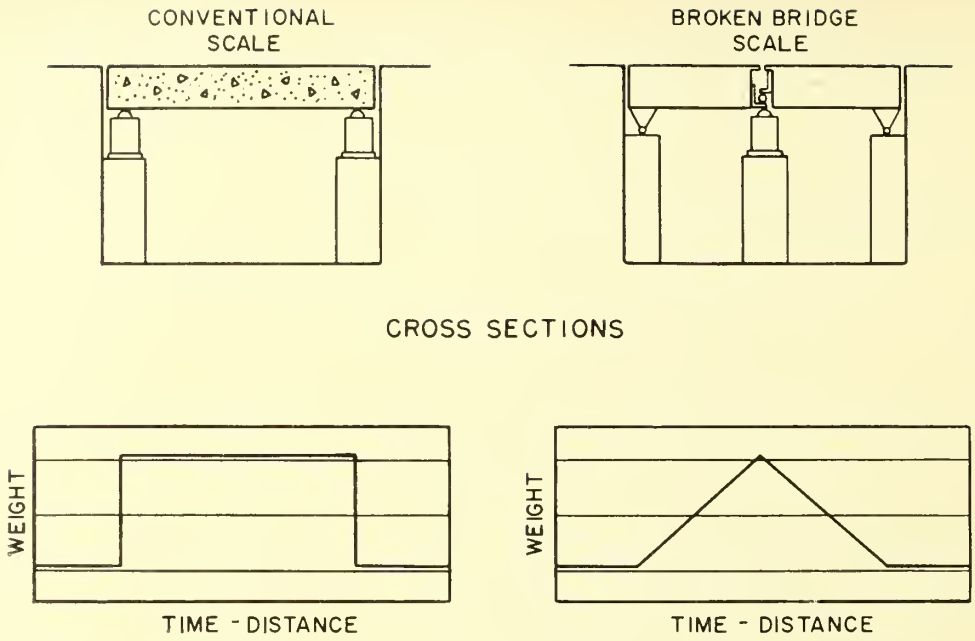

Figure 3.—Theoretical waveforms of output signals from two dissimilar scales.

Dynamic Weight Variations

The motion of a vehicle as it rolls along a modern highway will be affected by many factors. The condition and type of its suspension system, the amount and placement of its load, its speed, the type of body construction and its shape, wind velocity vectors, and other factors will influence its motion. Its motion in a vertical plane will become apparent on the road surface as variations of the weight applied to the surface by the wheels. As the vehicle's weight is the important factor in an in-motion weighing system, any variations of its weight from some static weight must be recognized. The dynamic weight variations will occur in some sinusoidal form, and in a rather narrow range of frequencies. Its frequency of variation will depend upon the physical limits of the suspension system and upon other physical factors related to the construction and loading of the vehicle. Typical values of the sinusoidal variations of dynamic weight of commercial trucks are in the range from 2 to 10 c.p.s. (*4*). The maximum variation of the dynamic weight of a given axle from its static value is as much as 100 percent heavy for lightly loaded vehicles and as much as 40 percent, heavy and light, for truck axles loaded at 14,000 pounds (*5*).

Assuming a pure sinusoidal variation of dynamic weight for a moving vehicle and as much as 50 percent variation from its static weight, the data presented in figure 5 show how the dynamic weight will vary with distance as the vehicle moves at different speeds. This result suggests the need for a weight-measuring system that consists of more than one weight-sensor platform, in order to increase the correlation of measured dynamic and static axle weights. Of course, whether the need exists will depend upon the use of the system. If the system is to be used to measure an actual dynamic weight, regardless of how this weight may vary from the static weight, then a single measurement will provide the desired result. However, if a weight is desired from the system that can be interpreted so as to obtain the actual static weight, then a more elaborate system containing more than one weight sensor must be used.

The output of the Taller-Cooper scale (fig. 4) is a radical departure from the theoretical waveform. It is recognized as a measurement of the variation of weight during the time the vehicle was on the platform. As the platform is about 3 feet wide, a vehicle having a high frequency of variation in weight, and moving at slow speed across the platform, yielded a rapidly varying output from the scale. The broken bridge, however, yielded an output at its maximum response point that is representative of the dynamic weight at that point on the platform. The Kentucky project has determined that each platform will measure equally well the weight applied to it.

Potential Uses of Each Platform

To satisfy the need to collect and analyze large volumes of traffic data, highway engineers are using electronic methods. Electronic in-motion weighing systems have been studied for several years, and many agencies are concerned with the development of a model station that could be used to collect the necessary data for load studies and planning purposes.

The Research Organization for Roadbuilding in West Germany devised the broken-bridge platform for weighing moving vehicles. This group was interested in classifying the measured axle weights into weight-groups of about 1,000 pounds. The recorded data showed how many axles weighed between 7,000 and 8,000 pounds, 8,000 and 9,000 pounds, etc. The output waveform has a gradually sloping excursion to its maximum; an electronic counter can be arranged to sum all weight signals in each of the preselected weight-groups (*6*).

A similar scheme for classifying axle weights has been developed by the Department of Scientific and Industrial Research, Road Research Laboratory, in England. Their system consists of a platform mounted in the road and supported on four load cells. The output of the cells is fed to a reflecting galvanometer whose motion is proportional to the load applied to the platform. The optical system of the galvanometer is arranged to cause a light beam to traverse a linear array of photocells. The placement of the photocells determines the interval of a weight-group, so that any applied load is classified into predetermined weight-groups. Each photocell is connected to a switching circuit that controls a mechanical counter.

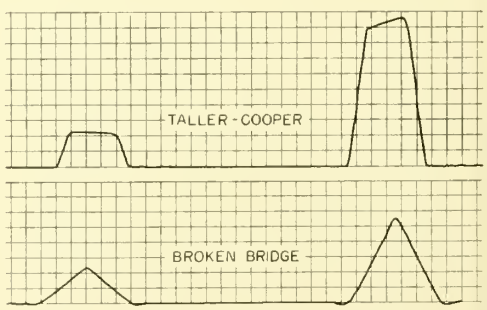

Figure 4.—Recorded output traces from Taller-Cooper and broken bridge scales.

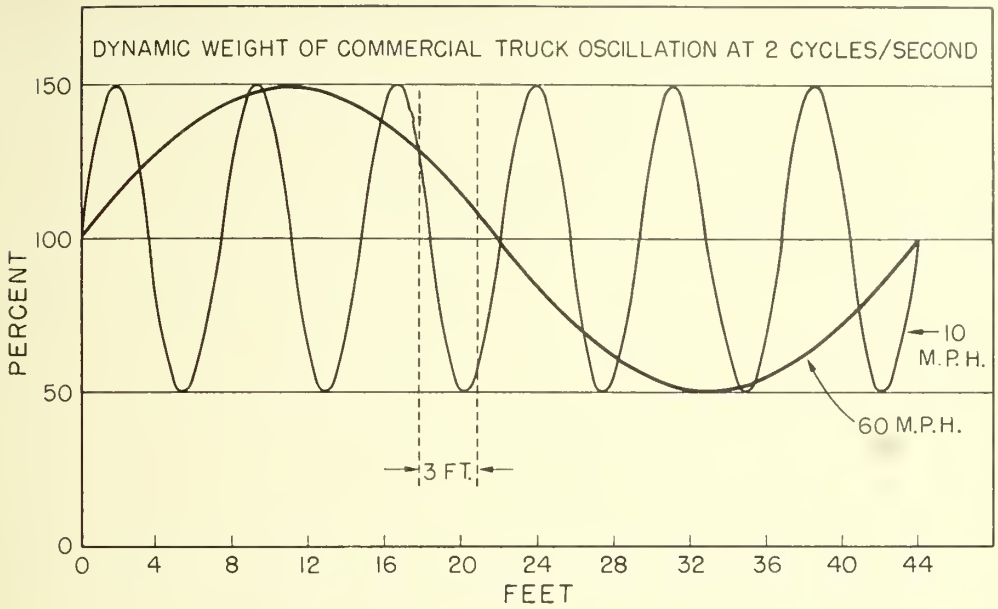

Figure 5.—Variation of dynamic weight of moving vehicle having 50 percent maximum variation from static weight, related to distance and speed of vehicle.

An excursion of the light beam past a given photocell causes its counter to register one count. Thus, the number of axle loads in any class is given by the difference of the readings of the two counters corresponding to the weight interval (*7*).

Each of these systems recorded data in digital form, but these data merely indicated how many axle loads occurred in a particular weight class. No information on actual dynamic weight variation of the axles was provided. To do this, the recording system must sample the load on an instantaneous basis rather than in discrete classification groups. An examination of the theoretical output waveforms of the Taller-Cooper and broken bridge scales shows that either type will permit such a sampling of loads applied to them. The rigid platform type of scale, however, is better suited for measuring actual dynamic weight variations of axles than the broken bridge type of scale because its output occurs over a relatively long period of time during which sampling could be done. The broken bridge type of scale yields a maximum value as an output, and this output will be representative of the dynamic load at that instant only. An electronic system to sample and hold this maximum output would require complex circuitry, and the cost would probably be prohibitive. But as the Taller-Cooper scale yields a varying output corresponding to the variations in load on the platform, the selection of one particular output value during this time as the dynamic weight of the axle is invalid. Perhaps an average of all output values occurring during the time the axle is on the scale would yield an acceptable weight result.

The Taller-Cooper scale was designed to measure loads applied to it in a nearly static state or while they moved slowly across the scale. Its response to fast moving loads can be improved through the use of large amounts of preload on the platform. The broken bridge scale was designed to be used in a measuring system where the loads would be moving at high speeds. Each of the two types of platforms have certain advantages and disadvantages and each has proved to be capable of doing the job for which it was designed.

The output waveform from the Taller-Cooper scale is suited to automatic measurement and recording of loads applied to it. However, some interval of time must be provided in the use of this type of scale during which a weight measurement may be made and sampled. The broken bridge type of scale, because of its single discrete measurement of the load applied to it, has not proved to be very well suited for the automatic recording of weights. But the output of a broken bridge type of scale can be more useful for in-motion weighing than the other type if a human operator can scan its record of weight measurements in the form of the Sanborn paper charts that were used for the test reported in this article. The peak of the recorded trace can be easily recognized from these charts, but the actual variation of dynamic weight of a passing vehicle is not apparent on the broken bridge recording because of this scale's mechanical construction.

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Effect of Moisture on Bituminous Pavements in Rocky Mountain Areas

REGION 9
BUREAU OF PUBLIC ROADS

Reported by [1, 2] WILLIAM L. EAGER,
Regional Materials Engineer

Determination of the quality of the materials and type of mix that will produce a bituminous pavement that will be serviceable and that will reasonably withstand the effect of moisture, especially in areas having a climate similar to that in the Rocky Mountain area, is the ultimate goal of highway engineers. Mindful of the disintegration of bituminous pavements caused by the effects of water or moisture in the Rocky Mountain area, test data on different types of asphalts, aggregates, and additives used in pavement construction have been summarized and analyzed. Additional information is presented in the companion article, "Stability of Bituminous Pavements Related to Aggregate Characteristics," by R. E. Olsen. The data analyzed for this article were obtained from tests performed in Colorado, Utah, and Wyoming, and in the Bureau of Public Roads Laboratory, Denver, Colo.

The study reported here was made in an attempt to forecast the serviceability of materials and correlate laboratory test results with actual performance of bituminous pavements. The analyses were made from the results of immersion-compression tests conducted on different bituminous paving mixtures.

Road failures have been pronounced in road-mix paving materials but they have also occurred to a serious extent in roads constructed of hot plant-mix and asphalt concrete materials. It is suggested that the immersion-compression test be used for determining in advance the probable effect of water on compacted paving mixtures. The determinations reached in this study indicate that further studies to correlate laboratory data and pavement performance would be worthwhile in other areas where the effect of water on bituminous pavement is noticeable.

Nature of the Problem

MUCH HAS BEEN written on the effect of water on bituminous paving mixtures (*1, 2,*[3]) and this problem is certainly not limited to the Rocky Mountain area. Ideally, water would have no effect on the pavement, but all too often it has reduced adhesion between the asphalt and aggregate to the point that serious stripping has occurred, causing loss of pavement stability and often severe raveling of aggregate from the surface. Stripping has been defined as the loss of asphalt films from aggregate surfaces in the presence of moisture (*3*); and raveling, as the loss of aggregate particles from the surface of the pavement—usually caused by loss of adhesion between aggregate and asphalt. This action is more pronounced in the road-mix types, but it has occurred to a serious extent in the hot-plant mix and asphaltic concrete types of pavement.

The Rocky Mountain area is generally dryer than other parts of the country. It might, therefore, be expected that the problem would be less severe there than elsewhere. However, this normal lack of moisture may well be a reason that the effect of water is more pronounced when it does become available. Water has not been available to leach out deleterious fines or disintegrate soft particles in natural deposits of aggregate, as in areas of greater rainfall. Possibly because of this, degradation is common for many of the aggregates available in the area. Although this degradation seldom causes any significant increase in plasticity, it does undoubtedly contribute to the reduction in resistance of the pavement to the effects of water.

A possible cause of the problem has been the widespread use of local, on-the-job aggregate deposits, rather than commercially produced aggregates. Specifications have commonly been written to fit these local aggregate sources rather than to require a high quality standard aggregate.

In recent years, more specifically since the start of the Interstate program, a general upgrading of specification requirements excludes some aggregates that were formerly permitted. If the use of these is permitted, they are upgraded by improved processing—prewasting of fines and use of screening equipment that removes and wastes coatings on the aggregate and breaks down and wastes the softer aggregate particles—or by the addition of mineral filler or hydrated lime. Washing of aggregates for bituminous construction has not yet come into general use, but as specification standards become higher and high-grade aggregates become less available, this will be a logical development.

Early Experience

The first bituminous pavements constructed in rural areas of the Rocky Mountains were nearly all of the road-mix type; slow curing road oils and subsequently cutbacks of the medium curing (MC) type were used. These surfaces were commonly recognized to lack resistance to the effects of moisture, so seal coats were used as a deterrent. Many times, however, water got into the pavement—probably through capillary action or vaporization. Excess water in the mat was a common cause of its early failure. A study (*4*) of bituminous pavements in Utah and Colorado in 1949 showed that pavements containing moisture in excess of 2 percent failed, and that the amount of moisture was proportional to the percentage of aggregate passing the No. 200 sieve. It was concluded that 12 percent was the maximum amount of aggregate that should be allowed to pass through this sieve. The tests did not indicate the plasticity index (PI) of the mat aggregate, but it is probable that higher PI values were associated with higher moisture content and with pavement failures.

Effectiveness of seal coats

Although seal coats have been effective in preventing or retarding surface raveling, their use has not always been effective in preventing stripping within the mat and, in fact, occasionally has been detrimental. Because of the general use of seal coats on road-mix mats, this practice has been carried over into hot plant-mix and bituminous concrete surfaces as they came into general use. Only in recent years has there been any tendency to leave seal coats off these higher type pavements and often then with reluctance and a common feeling that they would be needed within a short time. All too often this feeling was borne out.

The seriousness of the problem was first brought forcibly to attention by the results of an experimental project constructed in Colorado in 1941. A number of experimental sections of road-mix bituminous surfacing were constructed, in which different sources and types of asphaltic materials and a local

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[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

[2] E. G. Swanson and M. S. Tilzey of the Colorado Department of Highways; Travis Cole of the New Mexico State Highway Department; J. M. Desmond of the Wyoming Highway Commission; W. J. Liddle of the Utah State Department of Highways; and W. F. Fitzer of the Bureau of Public Roads, Region 9 Materials Testing Laboratory, cooperated with the author in providing information used in the preparation of this report.

[3] Figures in parentheses indicate references listed on page 51.

aggregate of adequate quality (by the normally accepted standards at that time) were used. However, all sections quickly showed serious distress; rapid deterioration of the surface (raveling) occurred with the first rain. This necessitated prompt seal coating of all sections, which obscured any differences in the experimental sections. A subsequent laboratory study of the aggregate by the Bureau of Public Roads Materials Research Division in Washington (*5*) lead to the development of the immersion-compression test as now provided by *The Standard Method of Test for Effect of Water on Cohesion of Compacted Bituminous Mixtures* (AASHO Designation: T 165 and ASTM Designation: D 1075).

Asphalt stripping

Even prior to the Colorado experiment, some very unhappy experiences occurred with asphalt stripping. In the middle thirties, the use of open-graded bituminous mat was tried on direct Federal construction projects. Because most of the aggregate sources were gravel, to obtain the 100 percent crushed aggregate required, the natural pit fines were wasted and the required aggregate was produced by crushing the oversized gravel. This produced a mat of high mechanical stability and high air voids. To keep surface water out, the surface was choked with fines and seal coated. The aggregates were from a granitic source and, although surface water may have been kept out and the surface looked good, the asphalt was soon stripping badly from the coarse crushed aggregate fragments in the lower parts of the mat, and these aggregate particles were coated with water instead of asphalt. The general opinion then was that the seal was ineffective and allowed surface water to enter and soften the mat. This type of construction, therefore, was discontinued in favor of a return to use of the dense-graded type of aggregate. However, there was no assurance that free water actually entered the mat from the surface. The open grading, in combination with the much greater affinity of the aggregate for water than for asphalt, permitted conditions favorable to the development of stripping. Temperature differentials are extreme in this mountainous area, and these differentials could well have led to condensation of moisture at times in the large void space of the open-graded mat.

On one of the projects referred to above, the State used the wasted pit fines to build a dense-graded road-mix on an adjacent project. Results were excellent. This does not prove that open-graded, crushed aggregate mats are necessarily all bad and that dense-graded ones are all good, but rather that when hydrophilic aggregates are used the chances for moisture to enter and cause stripping and softening of the mat are much less as the density of the compacted mixture is increased.

The projects referred to above were all built before the development of the immersion-compression test (AASHO T 165) or the static-immersion test, *Standard Method of Test for Stripping Test for Bitumen-Aggregate Mixtures* (AASHO Designation: T 182), or any other commonly accepted test to measure the effect of water on the aggregate-asphalt combination. In addition, commercial chemical antistripping agents had not come into general use.

Summary and Conclusions

In summary, the bituminous pavements in areas of the Rocky Mountains have always lacked resistance to the disintegrating effect of water. When road mixes were replaced by plant mixes, the problem was reduced but still existed. Additives—both a chemical type and hydrated lime—have alleviated the problem, as have better construction procedures and closer quality control of the materials. Seal coats have been necessary and have helped to compensate for undesirable stripping and raveling, but the goal is to so design and construct asphaltic concrete pavements that they will not need seal coats. Before that goal is reached it will be necessary to know just what further changes can be made in design, materials, or construction procedures. Considerable improvement in reducing the stripping and raveling problems may be expected if close attention is applied to the following listed details:

- Determination should be made by laboratory tests of the probable action of the compacted paving mixture in the presence of water. On the basis of the study reported here, the immersion-compression test has been concluded to be the best laboratory test for this determination.
- On the basis of the results of the laboratory tests, it has been concluded that the unsatisfactory aggregates should be eliminated, improved by screening or washing, or the differences compensated for by the addition of suitable admixtures (chemical antistripping additives, hydrated lime, filler, or other aggregate sizes to improve gradation).
- It has been concluded that good procedures for the design of paving mixtures and close control of construction (particularly mixing times, temperatures, and compaction procedures—more concentrated rolling when the mixture is still warm enough for the rolling to be effective) are essential for construction of bituminous highways that will withstand the effect of water.

Conclusions drawn from test

Conclusions drawn from analysis of test results compiled in table 1 are, as follows:

- Asphalt cement pavements have higher stabilities and higher indices of retained stabilities than cutbacks. Part of this is, no doubt, because a higher percentage of asphalt cement than of cutback can be used.
- Use of hydrated lime as an additive also permits use of a higher percentage of asphalt, this probably accounts for some of the higher index of retained stability of bituminous pavements when the hydrated lime is used. Hydrated lime adds to both dry and wet stabilities. Wet stabilities, when lime is added, are often higher than dry stabilities, probably because of continued reaction between lime and aggregate fines during the wet soaking period.
- A higher percentage of asphalt used in a mixture produces a higher index of retained stability although dry stability may be reduced by the higher percentage of asphalt.

Immersion-Compression Test

For laboratory determination of the effect of moisture on dense-graded, hot plant-mixed, bituminous mixtures, many States rely heavily on the immersion-compression test and design mixtures that will have a wet stability of 70 or 75 percent of the dry stability. The 70 or 75 minimum index of retained stability [4] generally seems to be satisfactory for hot-mix or asphaltic concrete, although it is often difficult to achieve without some stabilizing admixture. Swanberg and Hindermann (*6*) recommended a minimum index of retained stability of 75.

Methods of making test

Results of immersion-compression tests performed by Colorado, Utah, Wyoming, and the Bureau of Public Roads laboratories are shown in table 1. The test specimens made in the Colorado laboratory were prepared with a kneading compactor—125 blows at 500 p.s.i. foot pressure. The other laboratories prepared test specimens by using the double plunger method in accordance with the *Standard Method of Test for Compressive Strength of Bituminous Mixtures* (AASHO: T 167).

For water-immersed specimens (24 hours at 140° F.), the three States and Public Roads determined strength according to the *Standard Method of Test for Effect of Water on Cohesion of Compacted Bituminous Mixtures* (AASHO: T 165).

In Utah the actual wet stability is considered to be of greater significance than the index of retained stability and, therefore, a minimum wet stability of 150 p.s.i. is specified. In Wyoming the same is accomplished by requiring a minimum dry stability of 250 p.s.i. and an index of retained stability of 70 (70 percent of 250 is 187 p.s.i., which is, therefore, the minimum wet stability). This approach has considerable merit because mixtures of rather clean, coarse-graded aggregates often have a good index of retained stability but may be too low in either dry or wet stability or density to make the best pavement. In addition, chemical additives sometimes reduce the actual dry stability value and increase the wet stability value. In other words, they bring the two values closer together, thereby increasing the index of retained stability. When this is so, the index alone does not represent a true measure of the nature of the mixture or of the effect of the additives.

In comparing indices of retained stability, the method of compacting and testing the specimens must also be considered. For example, in Colorado the specimens are compacted with a kneading compactor, such as is used for Hveem stabilometer tests. Specimens so compacted show significantly higher stabilities, as well as higher indices of

[4] The term index of retained stability is synonymous with the term index of retained strength.

Source of data	Asphalt		Additive		Compressive strength, 77° F.		Index of retained strength
	Penetration grade	Amount, percent	Type	Amount,[1] percent	Dry	Wet	
Colo. Projects:							
2721	85-100	5.3	None	0	456	158	30
2721	85-100	5.7	PC[2]	2	444	570	128
2721	85-100	6.0	HL[3]	1	378	515	138
43076	85-100	5.8	None	0	293	56	16
43076	85-100	6.3	HL[3]	1	284	286	101
Utah projects:							
169-SA-63	85-100	5.2	None	0		195	
169-SA-63	85-100	5.2	HL[3]	1		283	
97-SA-63	85-100	5.3	None	0		45	
97-SA-63	85-100	5.3	HL[3]	1		154	
Wyo. projects:							
2938	85-100	6.0	None	0	302	188	62
2938	85-100	6.5	do	0	309	223	72
2938	[4] 85-100	6.0	HL[3]	2	404	404	100
2938	[4] 85-100	6.5	HL[3]	2	313	363	116
2938	[4] 85-100	6.0	None	0	332	272	82
2938	[4] 85-100	6.5	do	0	315	296	94
5120	[4] 85-100	6.0	None	0	234	89	38
5120	[4] 85-100	6.5	do	0	258	95	37
5120	[4] 85-100	6.0	HL[3]	2	285	208	73
5120	[4] 85-100	6.5	HL[3]	2	312	204	65
5120	[4] 85-100	6.0	HL[3]	3	430	234	54
5120	[4] 85-100	6.5	HL[3]	3	425	225	53
5120	[4] 85-100	6.0	HL[3]	4	453	275	61
5120	[4] 85-100	6.5	HL[3]	4	442	309	70
5120	[4] 85-100	6.0	PC[2]	3	280	131	47
5120	[4] 85-100	6.5	PC[2]	3	290	143	49
5120	[4] 85-100	6.5	AF[5]	1	278	110	39
5120	[4] 85-100	6.5	AF[5]	2	283	124	44
5120	[4] 85-100	6.5	FA[6]	2	310	80	26
5120	[4] 85-100	6.5	FA[6]	3	307	94	31
5120	[4] 85-100	6.5	LWA[7]	2	248	67	27
5120	[4] 85-100	6.5	LWA[7]	3	243	86	35
62-521	[4] 85-100	6.0	None	0	413	102	25
62-521	[4] 85-100	6.5	do	0	418	159	38
62-521	[4] 85-100	6.0	HL[3]	2	366	255	70
62-521	[4] 85-100	6.5	HL[3]	2	344	235	69
62-313-317	[4] 85-100	6.5	None	0	296	177	60
62-313-317	[4] 85-100	7.0	do	0	305	204	67
62-313-317	[4] 85-100	6.5	HL[3]	2	342	310	91
62-313-317	[4] 85-100	7.0	HL[3]	2	334	313	94
62-313-317	[4] 85-100	6.5	Cement	2	331	283	86
62-313-317	[4] 85-100	7.0	do	2	305	253	83
62-313-317	85-100	6.5	None	0	299	194	65
62-2778	[4] 85-100	5.25	None	0	196	68	35
62-2778	[4] 85-100	6.25	do	0	193	105	55
62-2778	[4] 85-100	6.75	do	0	190	157	82
62-2778	85-100	6.25	do	0	195	82	42
62-2778	85-100	6.75	do	0	207	115	56
62-2778	[4] 85-100	6.25	HL[3]	2	222	179	80
62-2778	[4] 85-100	6.75	HL[3]	2	213	231	108

[1] Additives incorporated in the mixture in the laboratory test, as contrasted to those added to the asphalt at the refinery, which are designated by footnote 4.
[2] Portland cement—used in mix at percentage shown (by weight of aggregate).
[3] Hydrated lime—used in mix at percentage shown (by weight of aggregate).
[4] Asphalt fortified at refinery with antistripping additive, amount and type not known. Other asphalts contained no refinery-added additive.

retained stability, than specimens compacted by the standard, double plunger, direct compression method.

Improved Design and Construction Standards

In recent years, standards of design and construction of bituminous plant-mix and and asphaltic cement have been upgraded in the Rocky Mountain area and are now believed comparable to standards used elsewhere. Minimum densities are 95 percent of laboratory density (Marshall standard or kneading compactor density), which will include air voids of 2 to 6 percent. Sufficient asphalt is used to fill at least 75 percent of the mineral voids in the compacted mixture. Generally, this requires about 6 percent of 85–100 penetration grade asphalt. The index of retained stability in the immersion-compression test is 70 or 75 for Interstate and primary roads. Commonly, an additive is required to obtain these indices of retained stabilities, and both chemical additives and hydrated lime are being used for this purpose. The normal laboratory practice is to test the bituminous mixture without any additive and, if the minimum index of retained stability is not obtained, to use a chemical antistripping additive, hydrated lime, portland cement, or other additive. Portland cement as an additive generally is not as satisfactory as hydrated lime. In some mixtures, chemical additives produce better results than hydrated lime but, in others, the reverse is true. Results of many immersion-compression tests are listed in table 1 and are illustrated in figure 1.

Experience with Hydrated Lime and Other Treatments

Emil Swanson, Materials Engineer for the Colorado Department of Highways recently presented a paper *Use of Hydrated Lime in Asphalt Paving Mixtures* before the National Lime Association (7). Some of the test results were rather startling. For example, one mixture without hydrated lime in the admixture fell apart during immersion in water. The same mixture, containing 1 percent hydrated lime, had a dry stability of 482 p.s.i. and a wet stability of 442 p.s.i., for an index of retained stability of 92. This aggregate was not cured after the hydrated lime was added. When other aggregates were used, a curing period of 2 to 5 days was needed for the lime to react with the aggregate before it was mixed with asphalt. On one such job, the indices of retained stabilities were obtained as follows:

Percent added		*Index of retained stability*
0	hydrated lime	43
1	hydrated lime (no curing period)	85
1	hydrated lime (48-hour curing period)	117

In the proportion normally used (1 to 1½ percent of weight of aggregate), hydrated lime is considerably more expensive than chemical additives, which may be used in the proportion of 0.5 to 1.0 percent of the weight of asphalt. In these proportions, the cost of the mixture is increased about 45 cents a ton if hydrated lime

results for different bituminous mixtures

Source of data	Asphalt		Additive		Compressive strength, 77° F.		Index of retained strength
	Penetration grade	Amount, percent	Type	Amount,[1] percent	Dry	Wet	
BPR:							
Sample C	60–70	5.8	None	0	183	84	46
Sample C	85–100	4.9	do	0	206	70	34
Sample C	85–100	4.9	HSA [8]	1	143	70	49
Sample C	85–100	4.9	HL [3]	1	277	130	47
Sample C	85–100	5.6	None	0	276	132	47
Sample C	85–100	5.6	HSA [8]	1	250	109	44
Sample C	85–100	6.2	HSA [8]	1	268	120	45
Sample C	85–100	6.3	HL [3]	1½	296	296	100
Sample D	85–100	5.6	None	0	227	226	100
Sample D	85–100	5.6	HSA [8]	1	212	238	112
Sample D	85–100	5.6	HL [3]	1	225	252	110
Sample E	MC–3	3.9	None	0	70	0	0
Sample E	MC–3	3.9	HSA [8]	1	99	40	40
Sample E	MC–3	5.0	HL [3]	1	95	46	48
Sample E	120–150	5.8	None	0	291	0	0
Sample E	120–150	5.8	HSA [8]	1	290	327	113
Sample E	120–150	6.7	HL [3]	1	315	235	75
Sample F	MC–3	5.2	None	0	62	13	29
Sample F	MC–3	5.2	HSA [8]	1	35	25	45
Sample F	MC–3	5.6	HSA [8]	1	65	27	42
Sample F	120–150	5.2	None	0	211	152	72
Sample F	120–150	5.2	HSA [8]	1	234	194	83
Sample F	120–150	5.6	HL [3]	1	262	219	84
Sample H	120–150	5.3	None	0	374	139	37
Sample H	120–150	5.3	HSA [8]	1	405	307	76
Sample H	120–150	6.1	HL [3]	1	387	267	95
Sample I	MC–3	4.0	None	0	148	33	22
Sample I	MC–3	4.0	HSA [8]	1	111	67	60
Sample I	MC–3	4.0	HL [3]	1½	184	130	72
Sample I	120–150	5.2	None	0	366	298	81
Sample J	MC 3	4.7	None	0		0	0
Sample J	MC–3	4.7	HSA [8]	1	102	55	54
Sample J	MC–3	4.7	HL [3]	1	118	64	54
Sample J	MC–3	4.7	HL [3]	1½	147	94	64
Sample K	MC–3	3.7	None	0	126	33	26
Sample K	MC–3	3.7	HL [3]	1	187	130	70
Sample K	120–150	5.7	None	0	259	131	51
Sample K	120–150	5.7	HL [3]	1	286	225	79
Sample L	MC–3	4.4	None	0	175	15	9
Sample L	MC–3	4.4	HSA [8]	1	192	35	18
Sample L	120–150	5.4	None	0	297	165	56
Sample L	120–150	5.4	HSA [8]	1	256	181	71
Sample L	120–150	5.4	HL [3]	1	293	192	66
Sample M	120–150	4.9	None	0	553	443	80
Sample M	120–150	4.9	HSA [8]	1	472	443	94
Sample M	120–150	4.9	HL [3]	1	518	468	90

[5] Asbestos fiber—used in mix at percentage shown (by weight of aggregate).
[6] Fly ash—used in mix at percentage shown (by weight of aggregate).
[7] Light weight aggregate—used in mix at percentage shown (by weight of aggregate).
[8] Concentrated heat stable additive—used in asphalt at percentage shown (by weight of asphalt).

is added, contrasted to about 11 cents a ton if the chemical additive is used. However, if the hydrated lime performs more satisfactorily than the chemical additive then, obviously, the additional expenditure is justified. In such a situation the proper cost comparison is between the added cost of the hydrated lime and the added cost of some other aggregate not needing the hydrated lime.

If a seal coat would definitely prevent the undesirable characteristics of a hydrophilic aggregate from manifesting themselves, then it would be proper to compare the cost of the additive or other treatment with the cost of a seal coat. Seal coats cost about 11 cents per square yard or, on a 3-inch thick mat, an additional 1.7 cents per ton of mixture. This, of course, is significantly cheaper than either the chemical additive at 11 cents per ton of mixture or the hydrated lime at 45 cents per ton of mixture. The trouble is, the seal coat will not always solve the problem; also so many uncertainties are involved in seal coat construction that the results are somewhat of a gamble.

Correlation of Test Results and Pavement Performance

Little correlation between pavement serviceability and laboratory mixture studies based on the immersion-compression tests was reported by Goldbeck (*8*). However, the Colorado experimental project (*4*) previously referred to did show good correlation. Although it is logical to expect a mixture that has a high index of retained stability to be better than one that has a low index, other factors affect the resistance of the mat to moisture and sometimes may be of greater significance and may obscure the action taking place in the immersion-compression test. If the mat is so dense, so well mixed, or is so well sealed over—either by compaction, traffic in warm weather, or by a seal coat—that water does not enter it, then stripping, swelling, and loss of stability cannot occur. If water could definitely be kept out of the mat and off its surface, there would be no need for concern about stripping or raveling or loss of stability. Obviously, there can be no such assurance and, therefore, it is proper to take all possible precautions to prevent mat damage by water.

Although no exact correlation exists between immersion-compression test results and the stability of the road (as stated previously, other factors affect the serviceability of the pavement), there are many examples of stripping, raveling, and even softening of the mat that are obviously caused by the effects of water in the mixture. The use of chemical additives or hydrated lime will not eliminate all these problems, as such materials cannot compensate for an inadequately designed or constructed pavement. Experience has shown, however, that such additives do help.

Some of the reasons why bituminous pavements are or are not damaged by water are known. An increasing number of pavements are little damaged even though they are not sealed. However, they still seem to be in the minority, and the number that last as long as

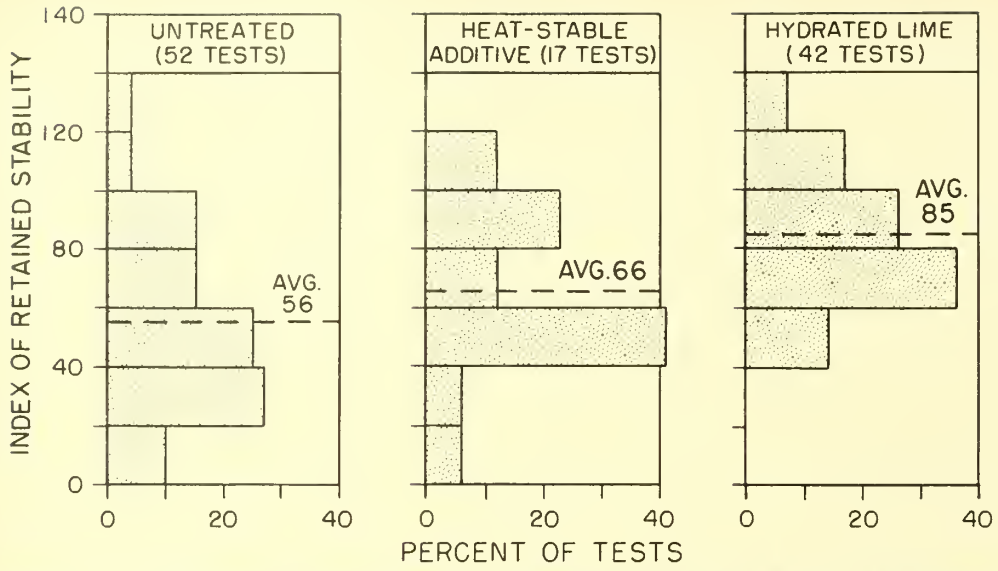

Figure 1.—Comparative results of tests in which additives were used in bituminous materials.

10 years, or even 5 years, without a seal coat does not constitute a large proportion of the bituminous pavements constructed.

Need for sealing

Whether all roads that are sealed actually need sealing is questionable. The need is often a matter of personal opinion, and some engineers and maintenance men might say a pavement needs sealing, whereas others, having a different background of experience, might not. Add the fact that the need for a seal coat usually develops at a time of the year when it is not possible to do anything about it (winter or spring), and it is understandable why some pavements are given precautionary seal coats when they might not need them. The first winter is usually the critical time; pavements seem to develop increased resistance to the effect of water as time passes, and pavement density is increased by traffic during warm weather. Thus, even if the value of an additive were lost over long periods of time, as has been argued, this might not matter so much if its value lasted through the critical early life of the pavement.

Late Season Construction

Generally, pavements placed late in the year require some form of sealing; but, those placed earlier, and therefore subjected to traffic during a season of warm weather, are much more likely to perform satisfactorily without being sealed. The possibility exists that compaction procedures need to be revised for pavement placed late in the year and that an effort should be made to duplicate, by rolling, the effect of traffic on the pavement during warm weather. Logically, this change in compaction procedure would be an increase in the amount of rubber-tired rolling while the mat is still warm. Although more rolling may be necessary to obtain the specified mat density, the specifications now do not require any different rolling procedures during cooler parts of the construction season. Density alone is not the criterion of a pavement that will adequately resist the effects of water. Rubber-tired traffic in warm weather tends to knead the pavement surface, working some asphalt mortar to the surface in much the same manner as it works fresh concrete and brings concrete mortar to the surface. It is doubtful whether the effect of traffic in warm weather can be duplicated on the surface of pavements laid late in the season. For example, the lower part of asphalt paving material placed on a cold base will cool quickly while the top remains so warm as to prohibit heavy rolling. It is believed that considerable improvement can be made in procedures for cold weather compaction.

Seal Coats

Not all pavements require seal coats because of the effect of water on the aggregate-asphalt combination. Raveling often starts because many mats become dry and brittle as time passes. Probable causes of this condition include weathering or hardening of the asphalt or selective absorption of the asphalt into absorptive aggregates. Seal coats generally consist of a coarse-graded gravel or crushed rock cover aggregate having a maximum seal of about three-eighths to one-half inch, applied at about 20 pounds per square yard over a rapid curing (RC) cutback used at the rate of about 0.20 gallon per square yard. In most places, the results have been good, but there have also been numerous exceptions where, because of adverse weather, uncontrolled traffic, or a stripping type of cover aggregate, the chips have failed to adhere to the binder, and the result has been a black, shiny, sticky nonuniform surface, as shown in figure 2. On some pavements, a sand or sand-gravel cover aggregate has been used, but the results have generally been less uniform and satisfactory than a regular type chip seal although the sand or sand-gravel seals have waterproofed the surface effectively.

A third type of seal frequently used when weather is not suitable for applying a chip or sand seal (as when a mat has been completed late in the construction season) is the so-called fog seal or black seal. A dilute emulsion (SS–1) or a light RC or MC is used without cover aggregate. The seal must, of course, be applied at a very light rate or the result will be a slippery surface. The SS–1 has an advantage in this regard because it may be diluted with water to any desired extent, to provide a complete yet very thin cover of

Figure 2.—Unsatisfactory (excessively rich) seal coat.

asphalt. Rate of application is about 0.02 to 0.05 gallon of asphaltic residue per square yard.

Type of seal coat

Another type of seal that is coming into increased use is the plant-mixed seal in which a semi-open-graded, crushed cover aggregate having a maximum size of three-eighths inch is plant mixed with asphalt cement and spread by a regular paver at about 60 pounds per square yard, as shown by figure 3. The mix is very rich (6 to 7 percent asphalt) and must be mixed at a relatively low temperature to permit retention of the required thick film of asphalt. As with the mat itself, it may be necessary to use an antistripping asphalt additive or hydrated lime in the mixture to provide adequate resistance to stripping. If this is required, as in uncoated cover aggregates used in seal coats or surface treatments, the need for an antistripping agent is determined by the static-immersion test (AASHO Designation: T 182), which requires 95 percent asphalt retention. The data in table 2 show typical results in this test. Typical grading requirements for aggregates in plant-mixed seals are as follows:

Sieve size	*Percent passing*
⅜-in	100
No. 4	30–50
No. 8	15–30
No. 40	0–10
No. 200	0– 3

This type of seal, if a large quantity is required, or if the contractor is already set up for hot plant-mix work, becomes less expensive than the chip seal and is far superior in appearance and durability.

Another type of seal that is becoming increasingly accepted is the so-called slurry seal that uses a sand, mineral filler and a dilute SS–1 emulsion. This seal is very inexpensive, but the results have not been as consistently good as those obtained by use of the plant-mixed seal previously described.

Table 2.—Static-immerson test results

Project number and source of data	Additive		Asphalt		
	Type	Amount, percentage	Type	Source	Percentage stripped from aggregate
Utah, No. 1	0	0	SC–3	Unknown	85–90
	A	1	SC–3	Unknown	45–50
	0	0	RC–4	A	85–90
	A	1	RC–4	A	5–10
	0	0	RC–4	B	85–90
	A	1	RC–4	B	0
	0	0	[1] 120–150	Unknown	70–75
	A	1	[1] 120–150	Unknown	0
Utah, No. 2	0	0	SC–3	Unknown	75–80
	A	1	SC–3	Unknown	25–30
	0	0	RC–4	A	60–65
	A	1	RC–4	A	5–10
	0	0	RC–4	B	15–20
	A	1	RC–4	B	0
Utah, No. 3	0	0	RC–4	A	70–75
	A	1	RC–4	A	2–5
	0	0	RC–4	B	0
	A	1	RC–4	B	0
Utah, No. 6	0	0	SC–3	Unknown	65–70
	A	1	SC–3	Unknown	5–10
	0	0	RC–4	A	70–75
	A	1	RC–4	A	2
	0	0	RC–4	B	2–5
	A	1	RC–4	B	0
	0	0	[1] 120–150	B	5–10
	A	1	[1] 120–150	B	0
Utah, 182–SA–61	0	0	MC–3	B	90–95
	C	1	MC–3	B	20–25
	0	0	RC–4	A	30–35
	B	1	RC–4	A	0–2
	0	0	[1] 120–150	A	5–10
	B	1	[1] 120–150	A	0–2
	0	0	RS–1 anionic.[2]	Unknown	10–15
	0	0	RS–2 cationic.[2]	Unknown	0–2
	0	0	RS–2 anionic.[2]	Unknown	10–15

[1] Penetration grade.
[2] Asphalt emulsions.

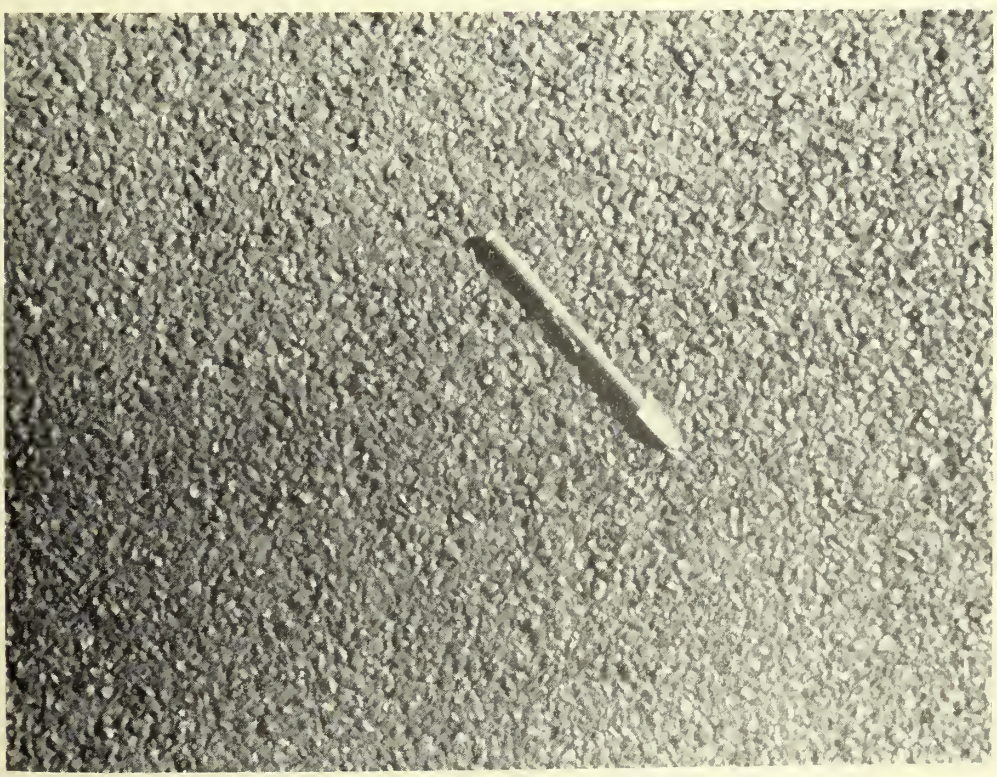

Figure 3.—Close-up of plant-mixed seal coat.

Other areas

Engineers from other parts of the country—particularly from farther east—often express surprise when mention is made of seal coating hot plant-mix or asphaltic concrete pavements. Their experience has lead them to expect that a properly constructed pavement will have adequate resistance to water and they, therefore, consider a seal coat to be entirely unnecessary. Even allowing for the possibility that some pavements get seal coated when they do not need it, a significant difference seems to remain between the water resistance of pavements in the Rocky Mountain area and those in some other areas. Just what causes this difference and what can be done about it poses a problem. Are the materials different, is something lacking in the construction procedures, or is the difference in climate a significant factor: Unquestionably, there are many hydrophilic aggregates in the study area, but this is probably true elsewhere as well.

Wyoming adopted new design criteria and specifications for plant-mixed surface for the 1962 season. The new specifications require: (1) the aggregate to be 100 percent crushed (including the fines), (2) a grading straddling a maximum density curve (0.45 exponential chart—(*9*)), and (3) a compacted density of at least 95 percent of Marshall specimens prepared according to the procedure outlined in the *Tentative Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus* (ASTM Designation: D 1559). The design criteria require a minimum index of retained stability of 70 for hightype plant mixture, 75 to 85 percent of voids filled with asphalt, and an air void content of 3 to 5 percent in the compacted pavement. Commonly, hydrated lime is needed as an additive to obtain an index of retained stability of the required 70 or more.

Some difficulty has been encountered in obtaining the specified 95 percent of Marshall density probably because of the high mechanical stability of the 100 percent crushed aggregate—and, also for the same reason, it has often been necessary to waste a very high percentage of the pit material in gravel deposits. This is not only expensive but, in some areas, there just is not enough aggregate available to permit such extravagent use. Consequently, the specifications have been revised to require only 50 percent of the material retained on the No. 4 sieve to be crushed, so most of the pit fines will now be used. Admittedly, the standards have thereby been lowered. The pit fines frequently contain undesirable portions of the deposit, but any undesirable characteristics will normally be corrected by the need to comply with the index of retained stability of 70.

Effect of PI

In a series of tests run on four typical Wyoming aggregates, engineers in the Bureau of Public Roads determined that aggregates having PI values of 5 and 6 gave indices of retained stabilities of only 55 and 60 (using Marshall specimens); whereas, when these same aggregates had their plastic fines removed by washing and they were replaced by limestone fines, the indices increased to 77 and 86. They also learned that if the proper minimum index of retained stability were 70, using standard 4- × 4-inch, double plunger, compacted specimens tested for unconfined compression, then the corresponding minimum for Marshall specimens would be 75. Aggregates have seldom been washed for use in bituminous pavements in Wyoming but, from these tests, washing seems to be one method of improving pavement quality.

The normally accepted limit of a PI of 6 for the fine portion of the aggregate proved to be too high for a great many mixtures, and the general opinion is that the specifications for plant-mix aggregates should require the fines to be nonplastic. Even when specifications do not require nonplastic fines, the specifications for an index of retained stability of 70 requires the addition of hydrated lime to most plant-mix aggregates that contain plastic pit fines.

Factors Influencing Immersion-Compression Results

The immersion-compression tests generally show that the indices of retained stability are higher when heavier asphalts are used and when more asphalt is used. The addition of hydrated lime permits the use of more asphalt and increases dry and wet stabilities. Wet stabilities when lime has been used in the paving mixture are often higher than corresponding dry stabilities, possibly because of continued reaction between lime and aggregate fines during the wet soaking period. Chemical additives are just as effective at increasing the index of retained stability when used with some materials as is hydrated lime, but the chemical additives do tend to reduce the dry stabilities.

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(*8*) *Immersion-Compression Tests Compared with Laboratory Traffic Tests on Bituminous Concrete*, by A. T. Goldbeck, in ASTM Special Technical Publication No. 94, *Symposium on Accelerated Durability Testing of Bituminous Materials*, 1950, pp. 53–66.

(*9*) *A New Graphical Chart for Evaluating Aggregate Gradations*, by J. F. Goode and L. A. Lufsey, The Association of Asphalt Paving Technologists Proceedings, technical sessions, 1962, vol. 31, pp. 176–207.

Stability of Bituminous Pavements Related to Aggregate Characteristics

Materials Research Division
Bureau of Public Roads

by[1] **ROBERT E. OLSEN,**
Highway Research Engineer

COMMENTS made by Mr. Olsen, in relation to the information on stability of bituminous pavements presented by Mr. Eager, were predicated upon results of tests made in the Bureau of Public Roads Laboratory, Washington, D.C. Mr. Olsen's comments were, as follows:

Mr. Eager has reviewed the problems associated with the use of local aggregates in bituminous construction in some of the Western States and reviewed several practices that are followed in their utilization. The Materials Research Division of the Bureau of Public Roads has been interested in and has followed State practices in bituminous construction in these States for several years and on a number of occasions has cooperated in studies.

A recent study was made to determine the relative quality characteristics of aggregates from four sources in the State of Wyoming. Some of the data collected pertaining to the physical characteristics of these aggregates are shown in table 1; these include aggregate gradation, sand equivalent test results, liquid limit and plasticity index results on the minus No. 40 sieve aggregate fraction and minus No. 200 sieve fraction, and hydrometer analysis of the minus No. 200 sieve material.

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

Table 1.—Physical characteristics of aggregates from Wyoming

	Aggregate source			
	P	PR	R	GC
Gradation of aggregate:				
Passing sieve size:				
¾-inch ... percent	100	100	100	100
½-inch ... do	90	88	94	90
⅜-inch ... do	84	71	85	78
No. 4 ... do	70	40	63	53
No. 10 ... do	55	25	48	37
No. 20 ... do	39	20	34	30
No. 40 ... do	28	17	25	21
No. 80 ... do	17	12	14	10
No. 200 ... do	7.6	8.0	9.4	5.9
Hydrometer analysis of aggregate:				
Passing No. 200 sieve ... percent	100	100	100	100
Smaller than:				
0.50 mm ... do	83	87	84	88
0.20 mm ... do	55	55	45	56
0.005 mm ... do	36	31	21	28
0.002 mm ... do	29	23	14	18
0.001 mm ... do	24	19	11	14
Clay in total aggregate [1] ... percent	2.7	2.5	2.0	1.6
Sand equivalent value	28	24	31	46
Plasticity index:				
Minus No. 40 sieve fraction	5	6	NP	NP
Minus No. 200 sieve fraction	21	13	8	8

[1] Calculated from percent finer than 0.005 mm.

Table 2.— Results of tests on bituminous mixtures made with aggregate from Wyoming

Characteristics of bituminous mixtures	Aggregate source			
	P	PR	R	GC
Aggregate not washed:				
Asphalt content, percent-mix basis	6.10	5.25	5.75	4.50
4-inch by 4-inch specimens:				
Compressive strength, dry ... p.s.i.	246	246	289	249
Compressive strength, wet ... p.s.i.	92	105	192	181
Index of retained strength	37	43	66	73
Volumetric swell ... percent	4.6	3.4	1.6	1.2
Air voids ... do	7.8	6.7	6.5	6.9
Marshall specimens:				
Stability, dry ... pounds	1,646	1,525	2,044	1,678
Stability, wet ... do	900	910	1,570	1,350
Index of retained stability	55	60	77	80
Air voids ... percent	4.3	4.3	3.9	3.7
Aggregate washed:				
Asphalt content, percent-mix basis	5.60	4.75	5.25	4.00
Percent dust after washing	1.8	1.0	1.3	0.8
Percent added limestone dust	2.0	3.0	3.4	2.2
Total passing No. 200 sieve ... percent	3.8	4.0	4.7	3.0
Marshall specimens:				
Stability, dry ... pounds	1,192	1,252	1,560	1,236
Stability, wet ... do	912	1,074	1,508	1,249
Index of retained stability	77	86	97	101
Air voids ... percent	6.3	4.8	4.3	4.8

One phase of the study included the determination of the effect of water on bituminous mixtures prepared with these aggregates. Test specimens were prepared at previously determined optimum asphalt contents and tested by the immersion-compression test for index of retained strength following the procedure given in ASTM Methods D 1074 and D 1075. The immersion period was 4 days at 120° F. In addition, the effects of water on the stability of 50 blow Marshall specimens were determined for mixtures of the same composition and using an immersion period of one day at 140° F. The same asphalt, an 85–100 penetration grade, was used for all mixtures. The results of these tests and related physical characteristics of the molded specimens are shown in table 2.

A comparison of the data in tables 1 and 2 shows increasing indices of retained strength and decreasing percentages of volumetric swell with decreasing values of plasticity index of the material passing the No. 200 sieve and percent clay (material finer than 0.005 mm).

To further evaluate these aggregates and to isolate the effect of clay in the bituminous mixture, a series of tests was made with most of the naturally occurring dust, or material passing the No. 200 sieve, removed by washing. Limestone dust was then added in amounts required to bring the total percentage of minus No. 200 sieve material to equal one-half of that which the respective aggregates originally contained. The asphalt contents of these mixtures were reduced by one-half percent in order to ensure that the air voids would be high enough to allow water to enter the molded specimens and that the asphalt film thickness would not be so great, so as to protect the aggregate particles from the effect of water.

The results of tests of mixtures using the washed aggregates are also shown in table 2. It will be noted that the level of dry strengths is lower for the washed aggregate mixtures than the unwashed aggregate mixtures in each case; this, however, should be expected with the reduced dust and asphalt contents. The index of retained strength of each mixture, however, is from 25 to 50 percent higher than for the comparable mixtures using the unwashed aggregates. This supports Mr. Eager's statement that the washing of aggregate may be a logical step toward the up-grading of local aggregates.

Figure 1 shows graphically the relationship of percent retained strength of molded specimens to percent clay in the respective aggregates. The percent clay in the washed aggregates, as shown in the figure, is based on the percent passing the No. 200 sieve after washing and the percentage of material finer than 0.005 mm., as determined by the hydrometer analysis of the original fines. The percentages of clay in the washed aggregates are, therefore, not absolutely correct.

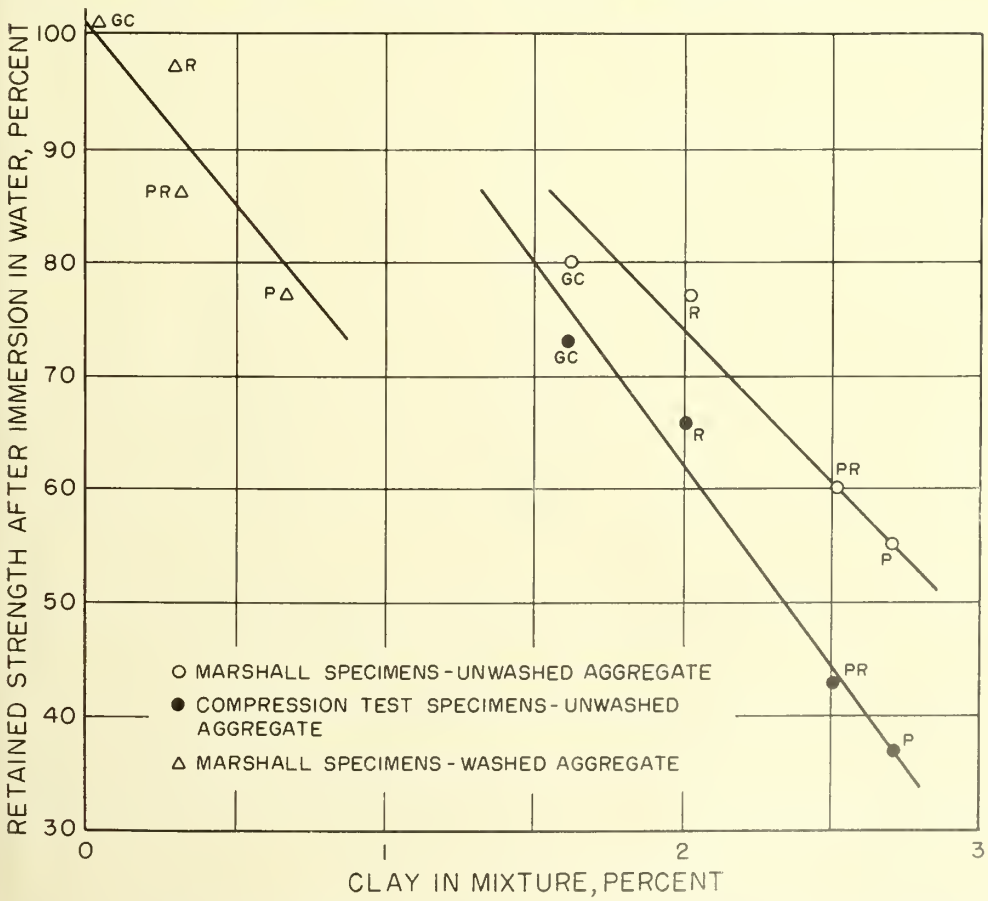

Figure 1.—Relationship of percentage of clay in bituminous mixtures and index of retained strength.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF MARCH 31, 1964

U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards, or improved to standards adequate for present traffic, built with Interstate or other public funds

MAJOR TOLL ROADS
Incorporated in the Interstate System

UNDER CONSTRUCTION

PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right-of-way acquisition completed or underway on many portions of these sections

Scale of map does not permit showing of status in urban areas and for very short sections

Preliminary Status or Not Yet in Progress	Engineering and Right-of-Way in Progress	Under Construction	Open to Traffic
6,526 Miles	12,349 Miles	5,457 Miles	16,668 Miles

34,474 Miles

INTERSTATE
TOTAL
41,000
MILES

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A JOURNAL OF HIGHWAY RESEARCH

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WASHINGTON

Exposure plot used to test durability of concrete specimens at the Bureau of Public Roads Laboratory

IN THIS ISSUE:

Long-time study of concretes prepared with portland cements

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IN THIS ISSUE

Natural Weathering of Concrete Specimens Prepared With Cements Used in the Long-Time Study, *by William E. Grieb and George Werner* 57

139 Million Drivers in 1980. *by E. M. Cope and Arlene R. Mundy* 68

New Publications 80

Errata 80

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Natural Weathering of Concrete Specimens Prepared With Cements Used in the Long-Time Study

Reported by WILLIAM E. GRIEB and GEORGE WERNER,[1] Highway Research Engineers

BY THE MATERIALS RESEARCH DIVISION BUREAU OF PUBLIC ROADS

In 1940 the Portland Cement Association, in cooperation with several consumer agencies, sponsored an investigation known as the Long-Time Study of Cement Performance in Concrete. Twenty-seven cements were selected for this study. They represented the principal cement producing areas of the United States and covered the range of chemical composition commonly encountered. This included all five ASTM types and air-entraining cements.

The major objective of the study was to provide a comparison of the performance, particularly with respect to durability, of concrete made with these cements. Comparisons were to be made to determine whether durability was related to certain properties of the cements, such as chemical composition or fineness.

Cements used in this investigation were used in an independent study by the Bureau of Public Roads. Boxtype concrete specimens made with these cements were exposed to outdoor weathering for more than 15 years. Some of the results of these tests were as follows: Concrete made with the Types II, IV, and V, and air-entraining cements were, in general, more durable than concrete prepared with Types I and III cements. Concrete prepared with cements having tricalcium aluminate (C_3A) content of more than 10 percent had poorer durability than concrete made with cements having C_3A contents of less than 10 percent.

Introduction

IN 1940 the Portland Cement Association sponsored a cooperative investigation known as *The Long Time Study of Cement Performance in Concrete.* The investigation was designed to develop information on the performance of 27 different cements in concrete subjected to different conditions of natural exposure at several locations. The five ASTM types of portland cement were included in the study. Information on the cements is quoted from the 10-year report (*1*):[2]

"The cements were made at representative cement plants under good operating conditions. Free lime in the clinker was uniformly low and the finished cements had autoclave expansions well below the limits permitted in ASTM and other specifications. A crew of trained observers was present to record data and take samples at various stages of the manufacturing process. Twenty-one clinkers were made and ground with gypsum to make 21 cements. In addition, 6 of the 21 clinkers were ground with gypsum and an air-entraining agent, making 27 test cements in all. There were eight cements of Type I composition (identified as Nos. 11 to 18, inclusive), five of Type II (Nos. 21 to 25, inclusive), three of Type III (Nos. 31, 33, and 34), four of Type IV (Nos. 41, 42, 43, 43A), and one Type V (No. 51). The six air-entraining cements are identified by the letter "T" following the cement number. (The term "air-entraining cement" had not come into use in 1939–40 when these cements were manufactured, and the letter "T" was used to indicate a "treated" cement.)"

Two of the Type IV cements, Nos. 43 and 43A, had different alkali contents, although produced by the same plant.

The Portland Cement Association and other cooperating agencies studied the behavior of the 27 cements in concrete subjected to different exposure conditions: (1) Test roads constructed in areas having severe, moderate, and mild climatic conditions; (2) piling exposed to sea water or fresh water, in severe or mild climates; (3) thin to moderate-sized concrete sections exposed to severe natural weathering; (4) specimens of *near-job-size* placed in exposure plots, in severe and mild climates; and (5) specimens placed in sulfate soils in mild climate. Many reports of the condition of the concrete for these projects have been published (*2*).

Summary

The following is a summary of the observations made on specimens prepared with cements used in the *Long Time Study of Cement Performance in Concrete.* The summary is based principally on the conditions of the boxtype specimens at the time of the 1961 examination.

- Concretes containing 6½ bags of cement per cubic yard and having water contents of less than 6.0 gallons per bag had excellent durability. Only 8 of 54 specimens in this category had evidence of serious distress after 16 years of exposure.
- Concretes containing 5½ bags of cement per cubic yard and approximately 6¼ gallons of water per bag had evidence of serious distress in 11 of 27 specimens after 16 years of exposure.
- Concretes containing 4½ bags of cement per cubic yard and more than 8 gallons of water per bag for most of the mixes had evidence of serious distress in 19 of 27 specimens after 12 years of exposure; 13 of the specimens had evidence of serious distress after only 3 years of exposure.
- The C_3A content of the cements appeared to have an appreciable effect on the durability of the concrete. Omitting the leanest mix of about 4½ bags per cubic yard, 18 of the 33 specimens prepared with cements having C_3A contents of more than 10 percent had evidence of significant to severe distress. Conversely, only one of the 48 specimens made with cements having less than 10 percent C_3A had evidence of significant distress.
- The use of treated (air-entraining) cements generally increased the durability of the concretes as contrasted with that of similar concretes prepared with the corresponding non-air-entraining cements. Although a number of the specimens prepared with the treated cements were given a poor rating, almost all of the disintegration causing such a rating occured in the basin of the boxes. Specimens prepared with cement No. 11T had the highest air content, averaging more than 10 percent, but had the second poorest average of durability for the treated cements.

[1] Mr. Werner retired from the Bureau of Public Roads in July 1964.

[2] References indicated by italic numbers in parentheses are listed on page 67.

Table 1.—Chemical properties of cements

[In percentages]

Cement No.	Silicon dioxide [1]	Aluminum oxide	Ferric oxide	Calcium oxide	Magnesium oxide	Sulfur trioxide	Loss on ignition	Free lime [2]	Sodium oxide	Potassium oxide	Chloroform soluble	Computed compound composition [3,4]				
												C_3S	C_2S	C_3A	C_4AF	$CaSO_4$
11	20.70	6.03	2.27	63.50	3.20	1.58	2.07	0.43	0.17	0.50	0.010	51	21	12	7	2.7
12	21.15	6.78	2.27	63.25	2.93	1.58	1.22	0.11	0.26	0.42	0.008	43	28	14	7	2.7
13	22.25	4.88	2.07	66.00	1.03	1.75	1.60	1.61	0.09	0.15	0.008	52	24	9	6	3.0
14	22.30	5.14	2.91	63.25	2.32	1.78	1.12	0.19	0.12	1.06	0.008	43	31	9	9	3.0
15	20.50	6.54	2.36	66.70	0.67	1.90	0.90	0.44	0.06	0.28	0.010	61	13	13	7	3.2
16	21.30	5.22	3.38	64.75	1.87	1.61	1.39	0.73	0.22	0.46	0.010	54	20	8	10	2.7
17	21.40	6.20	3.00	65.50	0.85	1.72	0.68	0.42	0.23	0.30	0.008	51	23	11	9	2.9
18	21.45	6.83	2.12	64.55	1.96	1.68	1.27	0.33	0.16	0.15	0.022	45	28	14	6	2.9
21	23.90	5.06	3.04	63.70	1.00	1.30	1.30	0.65	0.18	0.46	0.030	33	44	8	9	2.2
22	22.55	5.23	3.52	62.90	2.97	1.44	0.91	0.05	0.19	0.29	0.008	40	34	8	11	2.4
23	21.70	5.16	5.34	64.40	0.71	1.56	0.60	0.43	0.50	0.16	0.005	49	26	5	16	2.7
24	21.00	5.09	4.76	61.55	2.72	1.73	2.22	0.88	0.11	1.07	0.010	41	29	5	14	2.9
25	22.65	5.10	4.70	62.35	2.04	1.85	0.80	0.21	0.19	0.52	0.015	35	39	6	14	3.1
31	20.55	5.61	2.09	64.40	3.02	2.15	1.70	1.45	0.22	0.32	0.010	53	19	11	6	3.7
33	20.25	5.89	2.36	65.55	1.32	2.25	1.70	1.83	0.20	0.37	0.012	56	16	12	7	3.8
34	20.50	4.59	3.11	65.60	2.19	1.81	1.69	2.27	0.28	0.30	T	62	12	7	10	3.1
41	22.80	4.98	4.77	59.85	2.81	1.90	1.91	0.44	0.08	1.01	0.018	23	48	5	14	3.2
42	26.40	3.15	2.55	63.35	1.63	1.52	0.88	0.21	0.24	0.28	T	27	55	4	8	2.6
43	23.30	5.52	4.33	61.85	1.34	2.02	0.58	0.12	0.99	0.14	0.022	25	48	7	13	3.4
43A	25.45	3.80	3.00	63.40	1.07	1.82	1.03	0.35	0.35	0.03	0.020	28	52	5	9	3.1
51	24.40	3.25	3.00	64.90	1.47	1.44	0.86	0.54	0.11	0.52	0.012	46	35	4	9	2.5
11T	20.70	6.08	2.22	63.65	3.22	1.61	1.69	0.47	0.19	0.53	0.020	51	21	12	7	2.7
12T	21.30	6.46	2.34	63.70	2.80	1.57	1.29	0.12	0.22	0.32	0.020	46	27	13	7	2.7
16T	21.65	4.98	3.42	64.60	1.83	1.56	1.39	0.75	0.21	0.46	0.035	53	23	9	10	2.7
18T	21.70	6.66	2.09	64.45	1.85	1.73	1.17	0.38	0.13	0.17	0.042	46	30	14	6	2.9
21T	23.85	4.71	3.24	64.25	1.10	1.34	0.96	0.60	0.19	0.42	0.038	38	40	7	10	2.1
33T	20.50	5.71	2.34	65.75	1.32	1.98	1.87	1.84	0.19	0.35	0.020	57	16	11	7	3.4

[1] Percentages are based on weight.
[2] Free lime percentages are those reported by Portland Cement Association.
[3] Corrected for free lime.
[4] C_3S: tricalcium silicate; C_2S: dicalcium silicate; C_3A: tricalcium aluminate; C_4AF: tetracalcium aluminoferrite; $CaSO_4$: calcium sulphate.

Table 2.—Physical properties of cements

Cement No.	Specific gravity	Specific surface	Autoclave expansion	Normal consistency	Merriman sugar test		Time of set		Mortar strength					Air in mortar (ASTM C-185)	
					Neutral	Clear	Initial	Final	Tensile (1:3) after—			Compressive (1:2.75) after—		Hand mix	Machine mix
									3 days	7 days	28 days	7 days	28 days		
		Sq. cm./g.	*Percent*	*Percent*	*Ml.*	*Ml.*	*Hr., Min.*	*Hr., Min.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*P.s.i.*	*Percent*	*Percent*
11	3.12	1,870	0.11	24.5	5	6	3,45	5,00	310	375	485	2,765	4,460	4.6	4.3
12	3.17	1,840	0.13	25.0	3	3	3,45	5,15	300	405	440	2,750	4,800	4.8	5.2
13	3.09	1,780	0.00	25.0	48	56	3,00	5,15	260	320	415	1,965	3,900	3.7	2.7
14	3.13	1,860	0.11	23.0	4	5	3,15	5,15	320	385	460	2,865	5,185	6.0	5.4
15	3.12	1,920	0.16	28.0	46	66	4,45	7,15	370	435	510	3,815	5,150	4.5	3.0
16	3.19	1,850	0.06	25.0	6	7	3,30	6,00	360	400	505	2,935	4,800	4.8	4.0
17	3.18	1,790	0.08	25.0	6	7	3,30	5,30	335	435	505	2,835	4,465	5.6	3.8
18	3.14	1,920	0.06	25.0	20	29	3,30	6,00	300	375	460	2,850	4,850	2.3	2.1
21	3.17	1,760	0.01	25.0	9	13	5,30	8,+	210	310	405	1,950	3,800	4.6	4.6
22	3.21	1,860	0.08	24.0	2	2	4,00	6,00	250	355	470	1,900	4,435	4.2	3.7
23	3.14	1,800	0.01	25.5	3	3	4,20	8,15	250	340	420	2,175	4,460	2.2	---
24	3.17	1,960	0.12	23.0	5	5	2,30	5,00	275	350	425	2,215	4,185	5.9	5.2
25	3.24	1,930	0.07	24.0	2	2	4,30	6,30	230	305	425	1,315	3,135	5.6	4.4
31	3.12	2,730	0.09	28.0	50	73	1,30	3,30	420	545	550	5,185	6,700	3.0	2.3
33	3.11	2,500	0.17	27.0	52	73	1,30	3,45	450	515	530	5,000	5,665	1.9	1.0
34	3.17	2,440	0.09	27.0	50	64	3,00	5,00	485	460	490	3,865	5,500	3.2	2.8
41	3.22	2,000	0.10	23.0	3	3	3,20	6,30	220	255	390	1,110	2,685	5.3	---
42	3.24	1,920	0.00	25.0	2	2	6,35	7,40	185	250	400	840	2,250	1.8	2.1
43	3.22	2,010	0.02	25.0	3	3	3,30	5,30	170	265	400	1,035	3,065	6.2	5.7
43A	3.21	2,000	0.00	24.5	7	8	3,15	5,15	165	240	420	885	2,765	4.8	5.4
51	3.19	2,000	−0.01	25.5	5	7	5,30	7,00	255	305	425	1,750	3,935	3.6	3.0
11T	3.14	1,900	0.14	25.0	4	5	3,45	5,15	265	320	415	2,350	3,850	17.3	17.3
12T	3.16	1,750	0.15	25.0	3	3	4,00	6,25	245	390	470	2,315	4,300	9.4	9.3
16T	3.19	1,730	0.04	25.0	4	4	4,00	6,00	285	350	465	2,310	3,750	11.9	12.1
18T	3.14	2,040	0.08	26.0	30	46	2,00	3,45	260	325	400	2,600	4,665	11.2	10.6
21T	3.21	1,840	0.02	24.5	24	34	5,30	8,45+	210	285	390	1,800	3,465	11.2	11.9
33T	3.11	2,520	0.19	27.5	51	74	3,15	4,00	395	440	470	4,090	4,700	12.4	12.6

In the concretes prepared with the treated cements, poor durability was associated with high C_3A content. The use of treated cements appeared to be of most benefit in the lean-mix concretes. The average rating for the treated cements of the lean-mix concrete, 4.5 bags per cubic yard, was much better than the average ratings for the corresponding untreated cements.

- Most of the specimens prepared with the Types II, IV, V, and the treated cements had excellent durability.
- The specimens prepared with the Types I and III cements generally had poorer durability than those containing the other types of cement. Concretes prepared with two of the three Type III cements and three of the eight Type I cements had very poor durability. However certain of the Type I cements, Nos. 12, 14, 16, and 17, furnished concrete of very good durability.

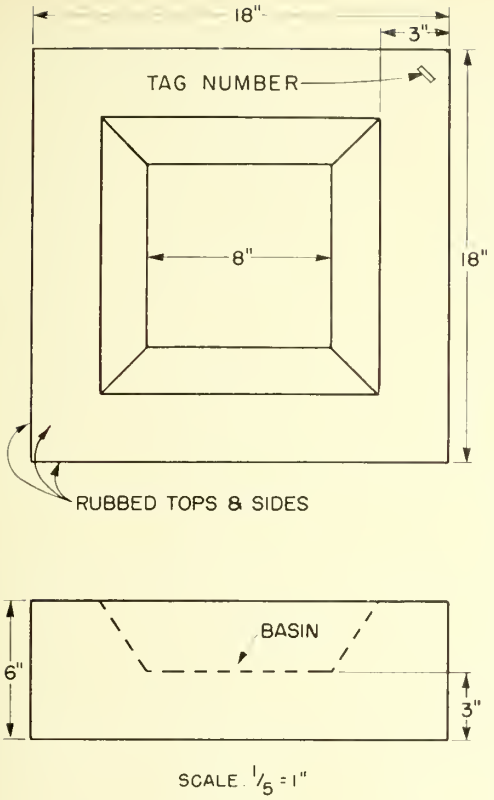

Figure 1.—Outdoor exposure box.

Participation by the Bureau of Public Roads

The Bureau of Public Roads cooperated with the Portland Cement Association in the regular inspection of several of the projects. In addition, samples of the 27 cements were sent to the Public Roads laboratory for special independent studies. The results of tests for the chemical and physical properties of these cements are shown in tables 1 and 2, respectively. The results of some tests differ slightly from those reported by other cooperating agencies (*2*, *3*).

Test specimens

Outdoor exposure tests of concrete specimens made with the 27 cements were an important part of the Public Roads study. Both boxtype and beam specimens were included. These specimens were placed in an exposure plot in 1945 and 1949. The box specimens were 18 inches square, 6 inches deep, and had a 3-inch deep basin. The basin was 12 inches square at the top; its sides were so tapered that it was 8 inches square at the bottom. The dimensions and shape of the box are shown in figure 1.

In preparing the box specimens, an 18-inch square wooden form was partially filled with concrete to a depth of 3¼ inches and rodded, and the surface was smoothed. The wooden core, which formed the basin of the box, was then placed on the smoothed surface. It was held in place by supporting strips nailed to the upper surface of the vertical sides of the wooden form. The mold was filled with concrete. The side slopes of the basin, as well as the vertical sides of the box, were spaded. The 3-inch top surface of the box was finished by careful screeding with a steel straightedge. After molding, a metal identification tag was inserted in a corner of the top surface.

Four groups of box specimens were made for each of the 27 cements. A set of three 3- by 4- by 16-inch beams, in which stainless steel gage studs had been cast in their ends, was made from the same batch of concrete as the box. These beams were cured in the same manner as the box specimens, and they were stored together in the outdoor exposure area.

The box and beam specimens were cast in an air-conditioned laboratory and left in their molds, which were covered with wet burlap for 48 hours, then removed from the molds and stored in moist air at 73° F. for approximately 28 days. They were then stored on the ground in an outdoor exposure plot adjacent to the laboratory. A photograph of the exposure plot appears on the cover of this magazine.

Table 3.—Grading and physical properties of aggregates

Grading and physical properties	Potomac River aggregate [1] used in rows 1, 2, and 3		White Marsh aggregate [2] used in row 4	
	Sand	Gravel	Sand	Gravel
Grading, percentage passing sieves:				
1 inch	------	100	------	100
¾ inch	------	75	------	75
½ inch	------	40	------	40
⅜ inch	100	15	100	15
No. 4	98	0	98	0
No. 8	85	------	81	------
No. 16	71	------	68	------
No. 30	48	------	53	------
No. 50	18	------	25	------
No. 100	4	------	6	------
Fineness modulus	2.76	7.10	2.69	7.10
Physical properties:				
Bulk specific gravity:				
Dry	2.59	2.58	2.63	2.64
Saturated surface dry	2.63	2.60	2.64	2.65
Absorption, pct	1.6	0.8	0.4	0.4
Strength ratio (Ottawa sand):				
Compressive strength:				
At 7 da., pct	113	------	97	------
At 28 da., pct	116	------	96	------
Tensile strength:				
At 7 da., pct	117	------	117	------
At 28 da., pct	114	------	112	------
Los Angeles wear test loss, pct	------	30.3	------	41.6
Accelerated soundness, Na_2SO_4 loss, pct	------	3.4	------	2.3

[1] Rounded and subangular quartz, and quartzite containing some chert.
[2] Rounded and subangular quartz, quartzite, and sandstone.

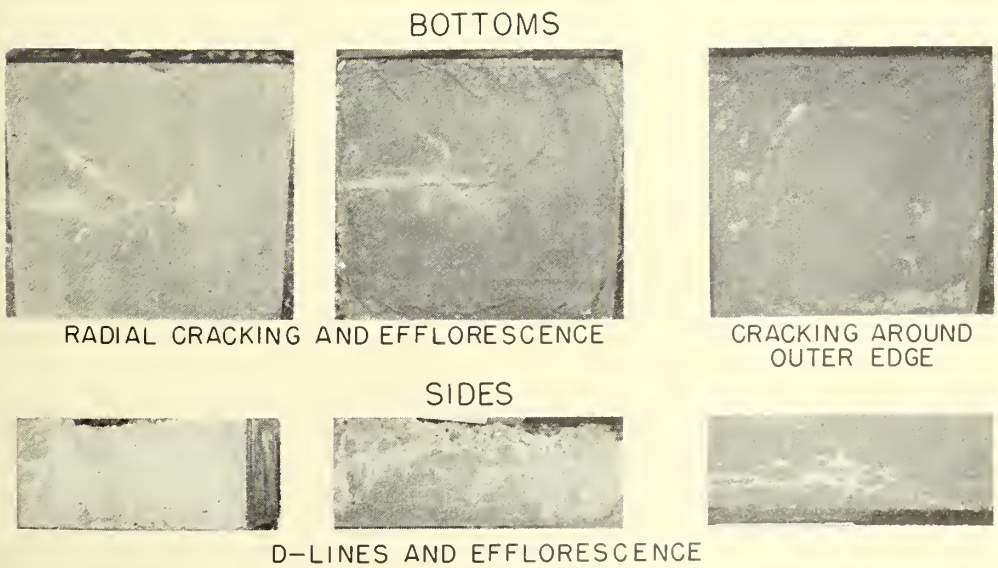

Figure 2.—Examples of distress in bottoms and sides of box specimens.

Each of the four groups of box and beam specimens was arranged in a separate row in the exposure plot. In the first three groups made, two adjacent side surfaces and the corresponding top faces of the boxes (fig. 1) were rubbed with a carborundum stone and water immediately after being removed from the molds, after 48 hours. The concrete was rubbed enough to break only the mortar surface and not deep enough to abrade the coarse aggregate. The rubbing developed a finish that was similar to that often used on handrails and other exposed surfaces of bridges. The other surfaces of the boxes were not rubbed. After about 4 years, all of the surfaces had weathered to a uniform sandy texture, and no difference between the rubbed and the unrubbed surfaces was apparent. The surfaces of the boxes in the last row were not rubbed.

The basin of each box was filled with pea-sized gravel and covered with water. The

only purpose of the gravel was to hold the water in the basins during windy weather. The temperature at the site of the exposure plot was considered moderate, averaging approximately 40 cycles of freezing and thawing per year. It is emphasized that deicing agents were not used in this study.

Concrete mixes

The aggregates used in making the concretes in the first three rows of specimens were a river sand and a gravel consisting mainly of rounded and subangular quartz and quartzite containing minor amounts of chert, micaceous schist, weathered granite, and gneiss. A bank sand and a gravel of rounded and subangular quartz, quartzite, and sandstone was used in making the concretes in the fourth row. Grading and other physical properties of the aggregates are listed in table 3.

The mix data for all four rows of specimens are listed in table 4. Nearly identical mixes were used in making the concrete specimens prepared with the same cements in the first and second rows. In these two rows, the cement content ranged from 6.3 to 6.6 bags per cubic yard of concrete, and the slump was from 4.7 to 6.6 inches. The sand content (percent of sand in total aggregate by solid volume) was 41 percent for the mixes prepared with the plain (non-air-entrained) cements. For those prepared with the treated (air-entraining) cements, the sand content ranged from 35 to 40 percent to obtain a uniform cement content. The weight of coarse aggregate per bag of cement was the same for all mixes in the first two rows. The water content ranged from 5.4 to 6.1 gallons per bag of cement for the concretes prepared with the plain cements and from 5.0 to 5.5 gallons for those made with the treated cements.

The air content ranged from 1.1 to 2.3 percent for the mixes prepared with the plain cements and from 4.0 to 11.2 for those made with the treated cements. The air content was determined by the use of a displacement pycnometer with an extension chimney. The method is identical in principle to the rolling procedure of ASTM Method C 173. The displacement method was used because no pressure air meter was available in the laboratory at the time these specimens were made.

The concrete in the third row was similar to that in the first two rows, except that the cement contents ranged from 5.4 to 5.7 bags per cubic yard of concrete, and the slump ranged from 4.5 to 6.1 inches. The water content ranged from 6.3 to 7.1 gallons per bag for concretes prepared with the plain cements and from 5.4 to 6.3 gallons for the treated cements.

The cement content of the concretes in the fourth row ranged from 4.4 to 4.8 bags of cement per cubic yard, and the slumps were from 7.0 to 8.5 inches. This mix was leaner than would be recommended for highway construction but was included so that failures might occur in a reasonable length of time. The water content ranged from 8.7 to 9.3 gallons per bag for concretes prepared with the plain cements and from 6.9 to 8.6 for those prepared with the treated cements. As previously stated, the fine and coarse aggregates used in this row were from a different source than those used in rows 1, 2, and 3.

Table 4.—Concrete mix data for four rows of specimens

Identification	Cement: Bags/cu. yd.	Water: Gal./bag of cement	Slump, concrete: Inches	Weight of plastic concrete: Lb./cu. ft.	Air, measured: Percent
ROW 1 [1]					
11	6.5	5.6	5.1	147.6	1.3
12	6.5	5.6	5.8	147.5	1.4
13	6.5	5.7	5.7	147.3	1.2
14	6.5	5.7	6.6	147.3	1.4
15	6.4	5.8	5.5	147.0	1.5
16	6.5	5.7	6.4	147.4	1.5
17	6.5	5.8	6.0	147.8	1.1
18	6.5	5.6	4.9	147.4	1.3
21	6.5	5.5	5.2	147.7	1.5
22	6.5	5.5	4.9	147.8	1.6
23	6.5	5.6	5.3	148.6	1.2
24	6.5	5.5	6.0	147.6	1.9
25	6.5	5.5	5.3	147.9	1.7
31	6.4	6.1	5.0	146.7	1.3
33	6.4	5.9	5.6	147.5	1.1
34	6.4	5.8	5.2	147.1	1.5
41	6.5	5.4	5.7	147.9	1.9
42	6.5	5.5	5.4	148.4	1.2
43	6.4	5.6	5.0	146.5	2.2
43 A	6.5	5.5	5.2	147.4	2.1
51	6.5	5.5	5.7	148.1	1.3
11 T	6.4	5.0	5.4	134.6	11.2
12 T	6.6	5.3	5.9	144.2	4.0
16 T	6.4	5.2	6.2	139.1	7.9
18 T	6.4	5.5	5.1	142.9	4.6
21 T	6.5	5.1	5.5	141.6	------
33 T	6.3	5.4	5.8	137.7	8.4
ROW 2 [1]					
11	6.5	5.6	4.8	147.2	1.8
12	6.5	5.6	5.2	147.3	1.6
13	6.4	5.7	4.8	146.8	1.3
14	6.4	5.7	5.5	146.7	1.9
15	6.4	5.7	5.0	146.6	1.4
16	6.5	5.6	5.4	147.2	1.5
17	6.5	5.7	5.5	147.3	1.4
18	6.5	5.6	4.7	147.8	1.3
21	6.5	5.5	4.9	147.6	1.5
22	6.5	5.5	5.0	147.6	1.5
23	6.5	5.6	5.9	148.1	1.1
24	6.5	5.4	5.7	147.2	1.9
25	6.5	5.5	5.5	147.5	1.6
31	6.4	6.1	5.2	146.6	1.4
33	6.4	5.9	5.3	146.5	1.3
34	6.4	5.8	5.5	147.0	1.4
41	6.5	5.4	5.6	147.3	2.0
42	6.5	5.5	5.0	147.8	1.3
43	6.4	5.6	5.7	146.6	2.3
43 A	6.5	5.5	5.8	147.0	2.1
51	6.5	5.5	5.1	147.8	1.3
11 T	6.4	5.0	5.5	134.7	10.9
12 T	6.6	5.3	5.4	144.2	4.0
16 T	6.4	5.1	5.7	139.0	7.9
18 T	6.3	5.5	5.0	142.4	4.6
21 T	6.4	5.1	6.0	140.6	6.6
33 T	6.3	5.3	5.4	138.1	8.0
ROW 3 [2]					
11	5.5	6.6	5.7	146.8	1.2
12	5.5	6.6	5.9	146.6	1.5
13	5.5	6.7	6.0	146.3	1.3
14	5.5	6.5	4.9	146.4	1.7
15	5.5	6.7	4.5	146.5	1.3
16	5.5	6.6	5.1	146.7	1.6
17	5.5	6.7	5.5	146.7	1.6
18	5.5	6.5	4.7	147.1	1.3
21	5.5	6.6	4.9	147.0	1.2
22	5.5	6.8	6.0	146.7	1.3
23	5.5	6.5	4.7	147.4	1.3
24	5.5	6.5	4.9	146.8	1.9
25	5.5	6.5	5.0	146.8	1.7
31	5.4	7.1	4.7	145.4	1.5
33	5.5	6.9	5.4	146.4	1.2
34	5.5	7.0	6.1	146.3	1.3
41	5.5	6.5	5.9	146.7	1.9
42	5.5	6.5	5.9	147.4	1.3
43	5.4	6.6	5.5	145.2	2.3
43 A	5.5	6.3	5.1	146.3	2.2
51	5.5	6.6	5.8	147.2	1.3
11 T	5.5	5.4	5.3	134.3	10.7
12 T	5.7	5.9	4.8	144.8	3.1
16 T	5.5	6.0	5.9	139.5	6.9
18 T	5.4	6.3	5.3	142.3	4.6
21 T	5.5	5.9	5.8	140.4	6.2
33 T	5.4	6.0	5.8	138.0	7.8
ROW 4 [3]					
11	4.4	8.7	8.0	146.1	1.0
12	4.4	8.7	7.0	145.7	1.2
13	4.5	8.8	8.0	146.3	------
14	4.5	8.8	7.5	146.8	------
15	4.5	9.0	7.5	146.5	------
16	4.4	8.8	8.0	145.7	1.0
17	4.4	9.0	7.0	145.9	------
18	4.4	9.2	7.5	146.5	0.7
21	4.4	9.1	7.5	145.9	0.8
22	4.5	8.8	8.0	146.9	------
23	4.5	9.0	8.5	147.1	------
24	4.5	8.8	8.5	146.3	------
25	4.4	8.8	7.0	145.6	1.3
31	4.4	9.2	7.0	145.4	0.7
33	4.4	9.3	8.0	145.3	0.8
34	4.4	9.3	8.5	145.9	------
41	4.4	9.0	8.5	146.0	------
42	4.5	8.9	8.0	146.8	------
43	4.4	8.9	8.0	144.5	------
43 A	4.4	8.9	7.5	145.1	------
51	4.5	8.9	8.0	146.9	------
11 T	4.8	6.9	8.5	137.2	7.6
12 T	4.5	8.1	7.5	143.7	3.3
16 T	4.6	7.5	8.5	138.2	7.5
18 T	4.4	8.6	7.5	141.2	4.2
21 T	4.6	8.0	8.5	141.5	4.6
33 T	4.6	7.9	8.0	140.5	5.2

[1] Aggregates used: Potomac River sand and Potomac River gravel.
Proportions (dry weight):
Plain cement, 94-192-276.
Air-entraining cement (11T), 94-150-276.
(12T, 16T, 33T, 21T), 94-171-276.
(18T), 94-184-276.

[2] Aggregates used: Potomac River sand and Potomac River gravel.
Proportions (dry weight):
Plain cement, 94-234-330.
Air-entraining cement (11T), 94-183-330.
(12T, 16T, 21T, 33T), 94-209-330.
(18T), 94-224-330.

[3] Aggregates used: White Marsh sand and White Marsh gravel
Proportions (dry weight):
Plain cement, 94-320-400.
Air-entraining cement (11T), 94-220-400.
(12T, 18T), 94-290-400.
(16T, 21T, 33T), 94-270-400.

Examination and Rating of Box Specimens

All box specimens were examined each year, generally during the summer months. Prior to an examination, the pea gravel and all loose particles of concrete were removed. The basin and all surfaces of the box specimens were thoroughly washed.

The basins were examined, and the extent and depth of scaling on their slopes and floor were recorded. The boxes were then set on a side, and their bottoms were inspected for evidence of cracks through the floor of the basin. Such cracks were generally outlined by efflorescence, and they appeared in radial form on the bottom of the boxes. A typical

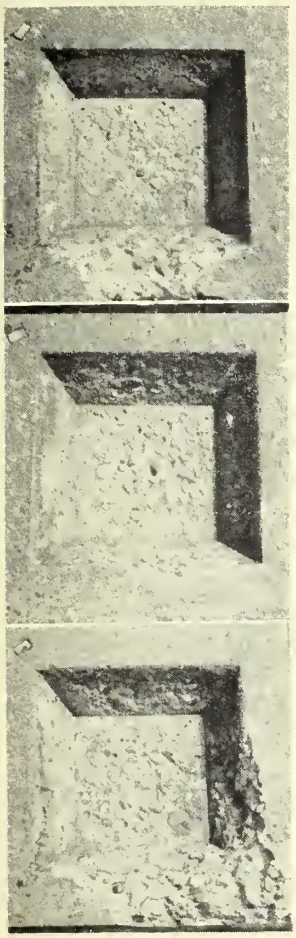

Figure 3.—Distressed concrete boxes in row 1; cement (C), rating (R); made Jan. 1945, photographed Aug. 1961.

Table 5.—Ratings of outdoor exposure boxes, 1955–61 inclusive

Cement, type and number	Row 1				Row 2				Row 3				Row 4			
	1955	1958	1960	1961	1955	1958	1960	1961	1955	1958	1960	1961	1955	1958	1960	1961
I:																
11	1	1	1	1	2	3	3	3	4	5	[1]6	7	[2]6	8	8	8
12	1	1	1	1	2	3	3	3	3	3	[1]4	4	1	2	2	2
13	1	1	1	1	1	1	2	2	2	2	2	2	[1]7	8	8	9
14	1	1	1	1	1	1	1	1	1	1	1	2	3	[1]4	4	4
15	2	[1]3	5	6	3	4	[1]4	5	[1]6	8	8	8	[2]8	9	9	10
16	1	1	1	1	1	1	1	1	2	2	2	2	[2]3	4	4	5
17	2	2	3	3	1	1	1	1	2	3	3	[1]4	2	3	3	3
18	3	3	[1]6	6	4	4	[1]6	7	[1]6	9	9	9	[2]9	10	10	10
II:																
21	1	1	1	1	1	1	1	1	2	2	2	3	[2]3	5	5	5
22	1	1	1	1	1	1	2	2	1	2	2	2	[2]6	7	6	6
23	1	1	1	1	2	2	2	2	2	2	3	3	[2]3	4	5	5
24	1	1	1	1	1	1	1	1	2	2	2	3	1	1	2	2
25	1	1	1	1	1	1	1	1	2	2	2	2	[2]3	4	4	4
III:																
31	4	4	3	3	4	4	3	4	[1]6	7	7	7	[2]9	10	10	10
33	4	[1]5	7	7	4	4	[1]6	7	[1]6	7	9	9	[2]9	9	9	10
34	2	2	2	2	2	2	2	2	2	2	2	2	[2]8	8	9	9
IV:																
41	1	1	1	1	1	1	1	1	1	1	1	1	[1]4	5	4	4
42	1	1	1	1	1	1	1	1	5	4	4	4	[2]2	5	5	5
43	1	1	1	1	1	1	1	1	1	1	1	1	[2]2	[1]5	4	4
43A	1	1	1	1	1	1	1	1	1	1	1	1	[2]2	[1]3	3	3
V:																
51	2	2	2	2	1	1	1	1	2	2	2	3	[2]2	5	5	5
Treated:																
11T	1	1	1	1	1	4	[1]5	4	4	[1]5	5	5	1	1	1	2
12T	1	1	1	1	1	1	1	1	1	1	1	1	1	[1]3	4	4
16T	1	1	1	1	1	1	1	1	2	2	2	2	1	1	1	1
18T	3	3	3	3	2	3	3	3	3	[1]4	4	4	2	[1]5	5	5
21T	1	1	1	1	1	1	2	2	2	2	3	2	1	1	1	1
33T	1	1	1	1	1	1	1	1	[1]5	5	5	5	1	1	2	2

[1] First evidence of leaking through concrete. [2] Leaking through concrete prior to 1955.

example of this condition is shown in figure 2. The boxes were then placed in the normal horizontal position, and their top and side surfaces were examined for D-lines,[3] crazing, cracking, scaling, or other sign of disintegration.

After an examination, the gravel was washed and again placed in the basins of the boxes. The gravel at the time of examination was almost free from mold or algae. Prior to 1955, the examination of the boxes consisted of noting scaling and signs of disintegration. From 1955 on, each box was given a numerical rating that, in the opinion of two examiners, represented its overall condition. The rating scale was from 1 through 10, inclusive. A rating of 1 indicated less than 10 percent light scaling in the basin and no other defects; whereas a rating of 10 indicated 100 percent deep scaling in the basin, radial cracks on the bottom, and disintegration of the top and sides of the box. A typical case for each value of the rating scale is described as follows:

Rating	*Condition of specimen*
1	Less than 10 percent light scaling in basin, no other defects.
2	5 to 50 percent light to moderate scaling in basin.
3	20 to 70 percent moderate scaling in basin.
4	40 to 90 percent moderate scaling in basin, plus slight radial cracks in bottom.
5	30 to 60 percent moderate to deep scaling in basin, plus radial cracks in bottom.
6	50 to 90 percent moderate to deep scaling in basin, plus radial cracks in bottom, plus few cracks in top and side surfaces.
7	50 to 90 percent moderate to deep scaling in basin, plus radial cracks in bottom, plus cracks or D-lines in top and side surfaces.
8	70 to 100 percent deep scaling in basin, plus radial cracks in bottom, plus numerous cracks or D-lines in top and side surfaces.
9	100 percent deep scaling in basin, plus radial cracks in bottom, plus top and side surfaces partly disintegrated.
10	100 percent deep scaling in basin, plus radial cracks in bottom, plus disintegration of top and side surfaces.

These descriptive examples are presented only for the purpose of illustration. Slightly different sets of distress conditions can result in the same rating. In terms of distress, a rating of 3 or less was considered to indicate superficial failure of the concrete; a rating of 4 or 5, significant failure; and a rating of more than 5, severe failure.

Each box was so placed in the exposure area that the metal identification tag was at the northeast corner. Examinations revealed that the eastern and southern surfaces showed, in general, more scaling than the other surfaces. The prevailing air currents were from the east and south.

Progressive Changes in Box Specimens

Prior to1951, no significant signs of incipient disintegration were noted in any of the box specimens. In 1951, an examination showed more than 10 square inches of scaling in the basins of the following listed boxes:

Row 1 (6½ bags, 5-inch slump)—Cements Nos. 33, 34.

Row 2 (6½ bags, 5-inch slump)—Cement No. 11T.

Row 3 (5½ bags, 5-inch slump)—Cements Nos. 15, 25, 51, 11T, 21T, 33T.

Row 4 (4½ bags, 8-inch slump)—Cements Nos. 11, 13, 18, 31, 33, 34, 41.

[3] D-lines, as defined by the Highway Research Board in its *Special Report 30, Pavement Condition Surveys*, are "A form of disintegration characterized by the successive formation of a series of fine cracks at rather close intervals paralleling edges, joints, and cracks, and usually curving across slab corners, the initial cracks forming very close to the slab edge and additional cracks progressively developing, each a little farther from the edge than the preceding one. Ordinarily the cracks are filled with a calcareous deposit."

Figure 4.—Distressed concrete boxes in row 2; cement (C) rating (R); made June 1945; photographed Aug. 1961.

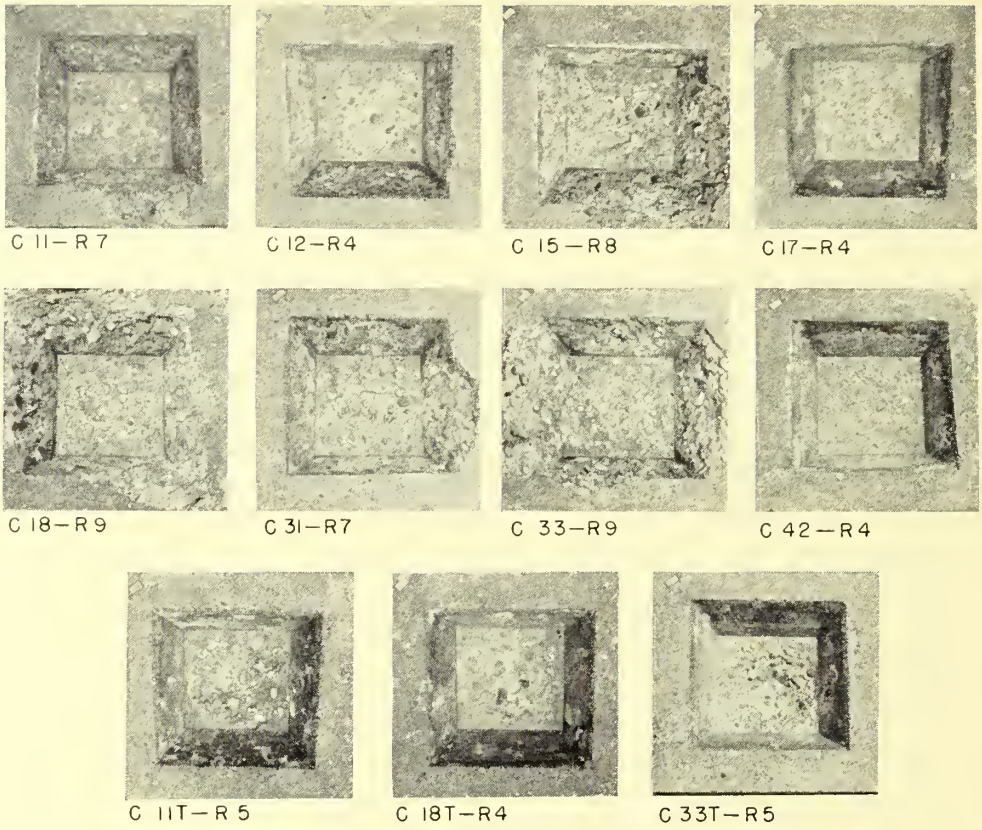

Figure 5.—Distressed concrete boxes in row 3; cement (C), rating (R); made Oct. 1945; photographed Aug. 1961.

The 1951 examination of the bottoms of the boxes showed no evidence of leakage through the concrete.

In 1952, more than 10 square inches of scaling was noted in the basins of the following listed additional boxes:

Row 1—None.
Row 2—Cements Nos. 18, 51.
Row 3—Cements Nos. 12, 31, 33, 42, 51, 18T.
Row 4—Cements Nos. 15, 16, 21, 22, 23, 25, 43.

In addition, 13 boxes from row 4 showed evidence of leakage through the concrete. These boxes were made with cements Nos. 11, 15, 16, 18, 21, 22, 23, 25, 31, 33, 34, 42, and 51. None of the specimens from the other three rows showed evidence of leakage. The first time that significant signs of disintegration were observed in any of the specimens was during this 1952 examination.

The boxes in row 1 were made in January 1945, and 5 days after being placed in the exposure plot they were subjected to freezing. They had received approximately 290 cycles of freezing and thawing at the time of the 1952 examination. The cement content of the concrete ranged from 6.3 to 6.6 bags per cubic yard, and the slump was from 4.9 to 6.6 inches.

The boxes in row 2 were made in June 1945, and were first subjected to freezing after about 5 months. At the time of the 1952 examination, they had been exposed to approximately 280 cycles of freezing and thawing. As stated previously, the mixes and slumps of the concretes in this row were nearly identical to those of the companion concretes in row 1.

The boxes in row 3 were made in October 1945 and were approximately 40 days old when first subjected to freezing. They had received about 280 cycles of freezing and thawing at the time of the 1952 examination. The cement content in row 3 was one bag less than that in rows 1 and 2, and the slumps of the concretes averaged about the same as those in rows 1 and 2.

The boxes in row 4 were made in September 1949 and were subjected to freezing after about 2 months. They had been exposed to 120 cycles of freezing and thawing at the time of the 1952 examination, when significant signs of incipient disintegration were noted. The concrete in this row contained 4.4 to 4.8 bags of cement per cubic yard, and the slump ranged from 7.0 to 8.5 inches.

Examinations in 1953 and 1954 revealed little progress in the disintegration of the box specimens. The ratings of the boxes from the 1955, 1958, 1960, and 1961 examinations are listed in table 5. This table also shows when cracks forming on the bottom of the boxes were first noticed.

Results of the Latest Examination of Box Specimens

The results of the latest examination of the box specimens, made in May 1961, comprise the principal part of this report. A summary of this examination is presented in table 6. The table shows the percentages and depths of scaling on the slopes and floors of the basins, together with any major defects or signs of disintegration on the bottom, side, and top surfaces of the box specimens.

Row 1. Table 6 lists the condition of the concrete boxes in row 1 according to the 1961 examination. The boxes had been exposed

to approximately 650 cycles of freezing and thawing. In general, they were still in excellent condition after more than 16 years of exposure. Only 3 of the 27 box specimens were given a rating of more than 3. These three boxes were prepared with cements Nos. 15, 18, and 33. Their condition is shown in figure 3. All had deep scaling in their basins, particularly on the basin floors. The bottoms of the boxes had radial cracks and the characteristic efflorescence. Areas of D-lines and efflorescence were also on their top and side surfaces, indicative of incipient disintegration. Each of these three cements contained more than 10 percent of tricalcium aluminate (C_3A).

Nineteen of the concrete boxes in row 1 showed no appreciable scaling or other defects and were given a rating of 1. All of the boxes prepared with the Type II and IV cements were in this group. The box made with treated cement No. 18T had deep scaling over 90 percent of the basin floor, but, because its other surfaces were free of defects, it was given a rating of 3. The air content of the concrete in this box specimen was 4.6 percent, and the C_3A content of the cement was 14 percent.

The average of the ratings for row 1 was 2.5 for the concrete boxes prepared with Type I cements, 1.0 for those made with Type II, 4.0 for those made with Type III, 1.0 for those made with Type IV, 2.0 for the one made with Type V, and 1.3 for those made with treated cements.

Row 2. The condition of the concrete boxes in row 2 are listed in table 6. The boxes had received 640 cycles of freezing and thawing at the time of the 1961 examination. Although of greater age before being subjected to freezing, the box specimens in row 2 were, on the whole, scaled slightly more than those in row 1. The reason for this difference is not known. Fourteen of the box specimens in row 2 were given a rating of 1, as compared with 19 in row 1. Five of the specimens were rated more than 3. These contained cements Nos. 15, 18, 31, 33, and 11T. The C_3A content of each of these five cements was more than 10 percent. Box specimens in row 1 that were prepared with cements Nos. 15, 18, and 33 were also rated more than 3.

The five specimens rated more than 3 showed deep scaling in their basins and evidence of incipient disintegration on their side and top surfaces. Four of the five specimens showed radial cracks and efflorescence on their bottoms. The condition of these box specimens is shown in figure 4. The box prepared with the treated cement 11T, which entrained 10.9 percent air in the concrete, was given a rating of 4. However, no tests were made to determine the size and spacing of the air voids in the hardened concrete. The 12 percent C_3A content for cement 11T might be a factor in causing the poor durability of the concrete.

The average of the ratings for row 2 was 2.8 for the box specimens made with the Type I cements, 1.5 for those made with Type II, 4.3 for those made with Type III, 1.0 for those made with Type IV, 1.0 for the one made with Type V, and 2.0 for those made with the treated cements.

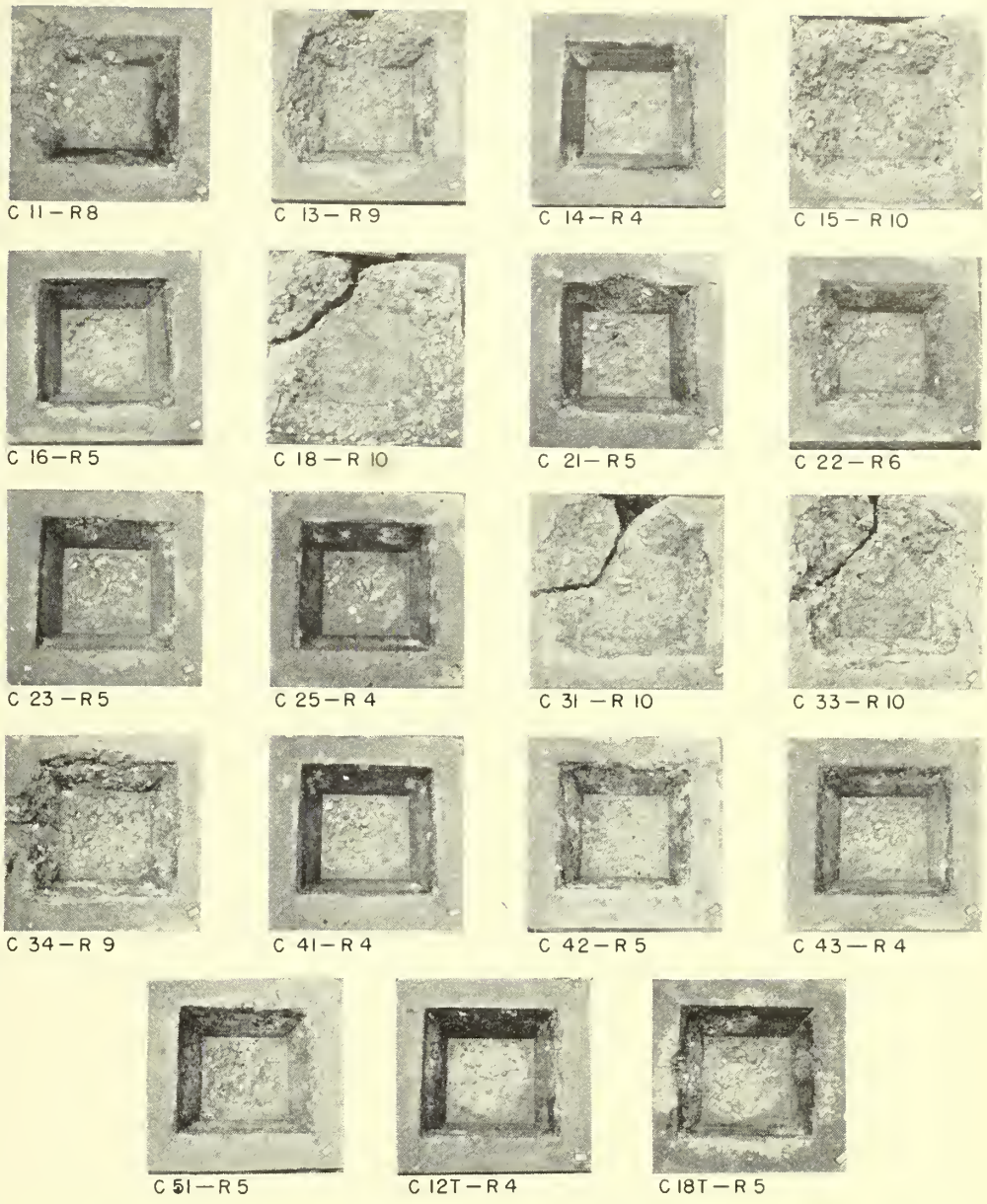

Figure 6.—Distressed concrete boxes in row 4; cement (C), rating (R); made Sept. 1949; photographed Aug. 1961.

Row 3. Table 6 summarizes the condition of the concrete boxes in row 3. Like those in row 2, they had received 640 cycles of freezing and thawing at the time of the 1961 examination. The specimens in row 3 were scaled considerably more than those in the first two rows. Only four of them were given a rating of 1. This group included three of the four Type IV cements and one of the six treated cements. Eleven of the 27 concrete boxes were rated more than 3. The condition of each is shown in figure 5.

The box specimens rated more than 3 were made with cements Nos. 11, 12, 15, 17, 18, 31, 33, 42, 11T, 18T, and 33T. All of these cements, except No. 42, had a C_3A content of more than 10 percent. All of the box specimens, except the one prepared with cement No. 42, had deep scaling over more than 40 percent of the surface area of the basin and had radial cracking and efflorescence on the bottoms of the boxes. The failure of the specimen made with cement No. 42 appeared to be structural. This concrete box had only moderate scaling over about 30 percent of its basin area. The bottom of the boxes had no sign of efflorescence, but it had D-lines around the outer edge of its bottom surface, as shown in figure 2.

Concrete prepared with three of the six treated cements had ratings of more than 3. These three cements, Nos. 11T, 18T, and 33T, had C_3A contents or more than 10 percent. The air contents of the concretes made with the above cements were 10.7, 4.6, and 7.8 percent, respectively, which indicates that air content alone did not ensure good durability for the conditions of exposure of the concrete boxes. In row 3, cement No. 12T was the only cement with C_3A content of more than 10 percent that produced concrete having good durability. The box specimen prepared with this cement had a rating of 1. The air content of the concrete was 3.1 percent.

Table 6.—Conditions and ratings of outdoor exposure boxes

Cement, type and number	Scaling in basin [1]		Box specimen [2]			Scaling in basin [1]		Box specimens [2]		
	Sides	Floor	Bottom	Sides	Top	Sides	Floor	Bottom	Sides	Top
	ROW 1 [3]					ROW 2 [4]				
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
I:										
11	5 L	None	No action	No action	No action	50 M	70 M	No action	Several cracks	Several cracks.
12	10 L	do	do	do	do	60 M	50 M	do	No action	No action.
13	2 L	do	do	do	do	15 L	10 L	do	do	Do.
14	None	do	do	do	do	None	None	do	do	Do.
15	50 D	70 M	8-in. dia., R.C., EFF.	50 D-lines, EFF.	50 D-lines, EFF.	80 M	90 M	6-in. dia., R.C., EFF.	do	10 D-lines, EFF.
16	5 L	None	No action	No action	No action	5 M	None	No action	do	No action.
17	60 M	60 M	do	do	do	None	do	do	do	Do.
18	80 M	90 D	4-in. dia., R.C., EFF.	75 D-lines, EFF.	75 D-lines, EFF.	90 D	100 D	9-in. dia., R.C., EFF.	40 D-lines, EFF.	50 D-lines, EFF.
II:										
21	None	None	No action	No action	No action	3 L	None	No action	No action	No action.
22	do	do	do	do	do	10 L	40 L	do	do	Do.
23	1 L	do	do	do	do	20 L	60 L	do	do	Do.
24	None	do	do	do	do	10 L	None	do	do	Do.
25	do	do	do	do	do	None	do	do	do	Do.
III:										
31	75 M	90 M	No action	do	do	90 M	90 M	do	5 D-lines, EFF.	15 D-lines, EFF.
33	80 D	80 D	15-in. dia., R.C., EFF.	70 D-lines, EFF.	70 D-lines, EFF.	90 D	80 M	12-in. dia., R.C.	70 D-lines, EFF.	90 D-lines, EFF.
34	20 M	80 M	No action	No action	No action	25 L	80 M	No action	No action	No action.
IV:										
41	None	None	do	do	do	None	None	do	do	Do.
42	do	do	do	do	do	do	do	do	do	Do.
43	do	do	do	do	do	do	do	do	do	Do.
43A	do	do	do	do	do	do	do	do	do	Do.
V:										
51	do	100 L	do	do	do	do	do	do	do	Do.
Treated:										
11T	do	None	do	do	do	do	90 D	8-in. dia., R.C., EFF.	Several cracks	Do.
12T	do	do	do	do	do	do	None	No action	No action	Do.
16T	do	do	5L	do	do	do	do	do	do	Do.
18T	do	90 D	No action	do	do	5 L	90 D	do	do	Do.
21T	do	None	do	do	do	10 L	40 L	do	do	Do.
33T	5 M	do	do	do	do	2 M	None	do	do	Do.
	ROW 3 [5]					ROW 4 [6]				
I:										
11	90 D	90 D	9-in. dia., R.C., EFF.	80 D-lines, EFF.	70 --lines, EFF.	70 D	90 D	15-in. dia., R.C., EFF.	30 D-lines, EFF.	{20 Disintegrated. 70 D-lines, EFF.
12	70 M	70 D	4-in. dia., R.C.	No action	No action	40 M	None	No action	No action	Light D-lines.
13	40 L	40 L	No action	do	do	90 D	100 D	16-in. dia., R.C., EFF.	100 D-lines, EFF.	{40 Disintegrated. 100 D-lines, EFF.
14	25 L	None	do	do	do	5 M	80 M	10-in. dia., R.C	No action	Light D-lines.
15	100 D	90 D	15-in. dia., R.C., EEF.	80 D-lines, EFF.	80 D-lines, EFF.			Disintegrated		
16	2 L	80 M	No action	No action	No action	None	80 D	16-in. dia., R.C., EFF.	1 crack thru	1 crack thru.
17	15 M	80 D	12-in. dia., R.C., EFF.	do	do	50 D	None	No action	No action	30 D-lines, EFF.
18	100 D	100 D	15-in. dia., R.C., EFF.	100 D-lines, EFF.	80 Disintegrated.			Disintegrated		
II:										
21	15 M	80 M	No action	No action	No action	30 M	80 D	16-in. dia., R.C., EFF.	1 crack thru	20 D-lines, EFF.
22	45 L	None	do	do	do	60 D	90 D	16-in. dia., R.C., EFF.	15D-lines, EFF.	{10 Disintegrated. 40 D-lines, EFF.
23	20 L	70 M	do	do	do	15 M	75 D	16-in. dia., R.C., EFF.	30 D-lines, EFF.	35 D-lines, EFF.
24	30 M	70 M	do	do	do	15 M	None	No action	No action	No action.
25	10 L	70 M	do	do	do	10 L	75 D	12-in. dia., R.C., EFF.	do	Do.
III:										
31	90 D	90 D	12-in. dia., R.C., EFF.	30 D-lines, EFF.	50 Disintegrated.			Disintegrated		
33	100 D	75 D	15-in. dia., R.C	100 D-lines, EFF.	80 Disintegrated.			Disintegrated		
34	70 M	None	No action	No action	No action	100 D	100 D	16-in. dia., R.C., EFF.	60 D-lines, EFF.	85 D-lines, EFF.
IV:										
41	None	None	No action	No action	No action	3 L	90 D	12-in. dia., R.C., EFF.	1 crack thru	Light D-lines.
42	25 M	40 M	20-percent D-lines.	30 D-lines	30 D-lines, EFF.	60 M	75 M	12-in. dia., R.C., EFF.	1 crack thru	10 D-lines, EFF.
43	None	None	No action	No action	No action	10 L	75 D	10-in. dia., R.C., EFF.	No action	Light D-lines.
43A	do	do	do	do	do	40 L	75 M	2-in. dia., R.C	do	Do.
V:										
51	25 L	100 M	do	do	do	40 M	90 M	12-in. dia., R.C., EFF.	1 crack thru	15 D-lines, EFF.
Treated:										
11T	10 M	90 D	8-in. dia., R.C., EFF.	do	do	10 L	None	No action	No action	Light D-lines.
12T	2 L	None	No action	do	do	5 M	75 D	12-in. dia., R.C., EFF.	do	Do.
16T	None	80 M	do	do	do	None	None	No action	do	No action.
18T	5 L	80 D	4-in. dia., R.C	do	do	50 M	90 D	10-in. dia., R.C., EFF.	1 crack thru	1 crack thru.
21T	None	80 D	No action	do	do	None	None	No action	No action	Light D-lines.
33T	do	90 D	12-in. dia., R.C, EFF.	do	do	15 M	do	do	do	Do.

[1] D, deep; M, moderate; and L, light.
[2] R.C., radial cracking; and EFF, efflorescence.
[3] Boxes made January 1945; rated May 1961. Mix: Cement factor 6.3—6.6 bags, slump 4.9—6.6 inches, water 5.0—6.1 gallons.
[4] Boxes made June 1945, rated May 1961. Mix: Cement factor 6.3—6.6 bags, slump 4.7—6.0 inches, water 5.0—6.1 gallons.
[5] Boxes made October 1945, rated May 1961. Mix: Cement factor 5.4—5.7 bags, slump 4.5—6.1 inches, water 5.4—7.1 gallons.
[6] Boxes made September 1949, rated May 1961. Mix: Cement factor 4.4—4.8 bags, slump 7.0—8.5 inches, water 6.9—9.3 gallons.

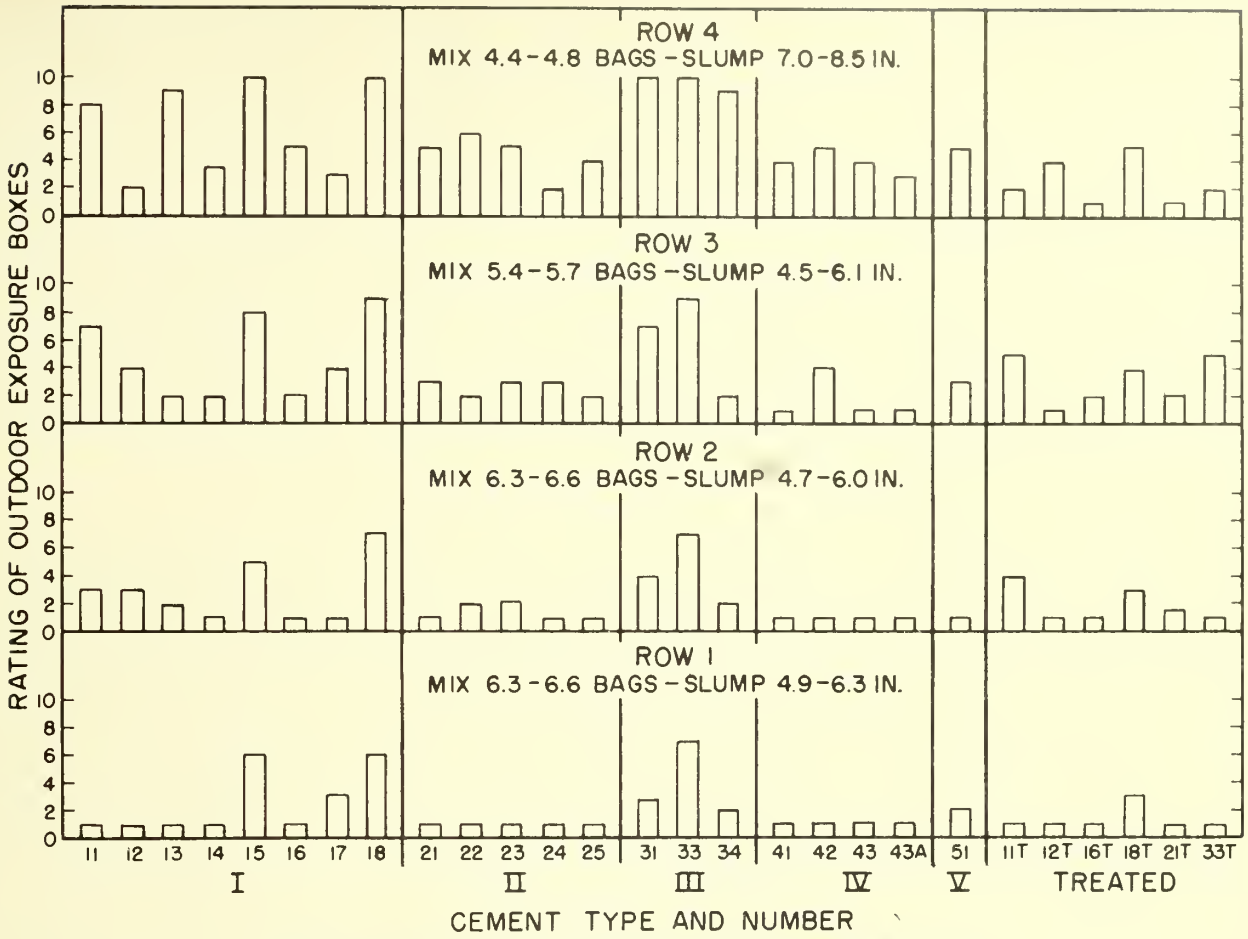

Figure 7.—Visual ratings of outdoor exposure boxes, May 1961.

Average ratings for the box specimens in row 3 were 4.8 for those made with Type I cements, 2.6 for those made with Type II, 6.0 for those made with Type III, 1.8 for those made with Type IV, 3.0 for that made with Type V, and 3.2 for those made with the treated cements.

Row 4. The condition of the concrete boxes in row 4 are presented in table 6. As mentioned previously, the concrete in row 4 was prepared with aggregates from a different source than the concretes in the other three rows. The box specimens in row 4 had been exposed to 480 cycles of freezing and thawing at the time of the 1961 examination.

As might be expected, the somewhat low cement content, high slump concrete in row 4 developed considerably more distress than the concretes in the other three rows. Only two of the box specimens in this row, the ones prepared with treated cements, Nos. 16T and 21T, were given a rating of 1. The four box specimens prepared with cements Nos. 12, 24, 11T, and 33T were rated at 2. Oddly, the specimen prepared with cement No. 11T, which was rated 4 and 5 in rows 2 and 3, was one of those rated as 2. Nineteen of the 27 box specimens in this row were given ratings of more than 3. No consistent relation existed between the ratings of these specimens and the C_3A content of the cements. The four concrete boxes that were given a rating of 10—Nos. 15, 18, 31, and 33—contained cements with C_3A contents of more than 10 percent. However, of the six concrete boxes that had a rating of 2 or less, three were prepared with cements having a C_3A content of more than 10 percent. Figure 6 shows the condition of the concrete boxes having a rating of more than 3.

For this row, the water content of the concrete appeared to have more effect on the durability than the chemical composition of the cement.

The average of the ratings for concrete in row 4 was 6.4 for the box specimens made with Type I cements, 4.2 for those made with Type II, 9.7 for those made with Type III, 4.0 for those made with Type IV, 5.0 for that made with Type V, and 2.5 for those made with the treated cements. The box specimens in row 4 prepared with the treated cements showed, in general, much better durability than those prepared with the corresponding untreated cements. This condition was not so pronounced for the other three rows. The beneficial effects of entrained air were more apparent for the lean concrete.

Table 7.—Summary of ratings of outdoor exposure boxes, rated May 1961

Cement, type and number	Rating for rows—			
	1	2	3	4
I:				
11	1	3	7	8
12	1	3	4	2
13	1	2	2	9
14	1	1	2	4
15	6	5	8	10
16	1	1	2	5
17	3	1	4	3
18	6	7	9	10
II:				
21	1	1	3	5
22	1	2	2	6
23	1	2	3	5
24	1	1	3	2
25	1	1	2	4
III:				
31	3	4	7	10
33	7	7	9	10
34	2	2	2	9
IV:				
41	1	1	1	4
42	1	1	4	5
43	1	1	1	4
43A	1	1	1	3
V:				
51	2	1	3	5
Treated:				
11T	1	4	5	2
12T	1	1	1	4
16T	1	1	2	1
18T	3	3	4	5
21T	1	2	2	1
33T	1	1	5	2

Table 8.—Change in N^2 in beams after 12 to 16 years of exposure[1]

Cement, type and number	Percent increase or decrease in N^2, in rows—			
	1	2	3	4
I:				
11	−11.0	−6.1	−14.5	−20.3
12	−7.2	−6.7	−6.7	−12.9
13	8.4	7.6	7.5	3.8
14	−10.5	3.3	−1.6	0.9
15	−7.2	3.5	2.2	[2] −46.1
16	1.7	3.8	3.3	6.9
17	−5.1	−1.8	−2.5	−8.2
18	1.3	2.9	0.3	[2] −51.8
Average	−3.7	0.8	−1.5	−16.0
II:				
21	10.5	−5.0	13.0	23.4
22	10.4	2.4	5.0	−10.4
23	10.4	5.2	7.2	−1.0
24	6.2	1.4	2.7	2.1
25	14.7	10.9	19.6	10.5
Average	10.4	3.0	9.5	4.9
III:				
31	−8.7	−5.1	−9.2	[2] −58.4
33	−6.6	0.5	−18.4	[2] −34.3
34	−1.0	1.8	4.9	[2] −45.3
Average	−5.4	−0.9	−7.6	−46.0
IV:				
41	8.3	8.7	23.6	20.5
42	17.3	18.9	37.4	34.4
43	9.8	5.5	12.5	18.4
43A	18.0	19.2	22.2	27.1
Average	13.4	13.1	23.9	25.1
V:				
51	13.4	13.9	19.3	0.6
Treated:				
11T	−15.7	−13.8	−6.5	8.1
12T	−5.8	−7.9	−8.4	−9.1
16T	1.9	2.5	2.5	10.5
18T	−2.0	−3.1	0.6	3.4
21T	7.5	6.0	12.6	21.5
33T	−0.6	4.1	1.6	7.0
Average	−2.4	−2.0	0.5	6.9

[1] Each percent is the average of tests on three 3- x 4- x 16-inch beams.
[2] Beams showed numerous cracks.

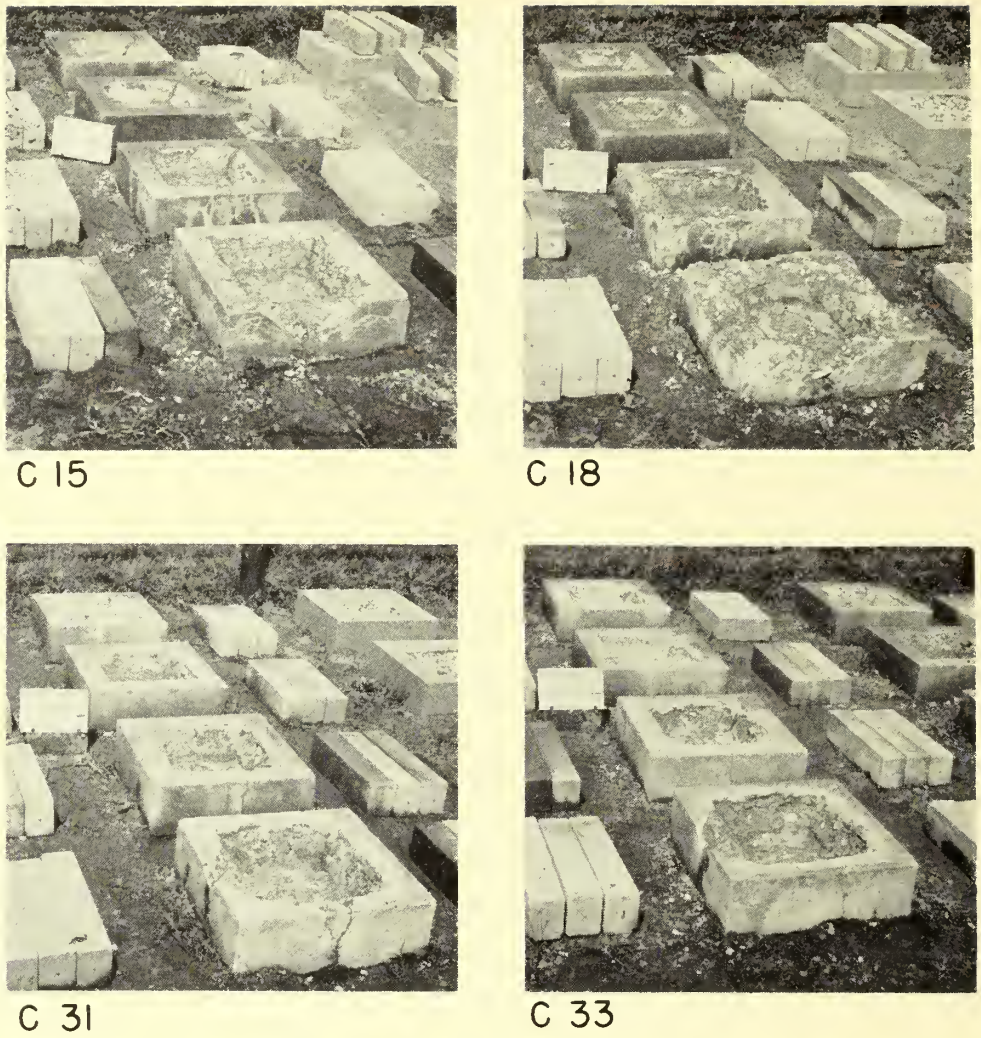

Figure 8.—Concrete boxes made with cements (C) having the poorest durability

Summary of All Four Rows

The ratings of the individual concrete boxes at the time of the last inspection are listed in table 7 and are shown in figure 7. Overall, the concretes prepared with Type III cements showed the poorest resistance to natural weathering. However, the concrete boxes prepared with cement No. 34, Type III, had good durability in all but the fourth row. The concrete containing the Type I cements had the next poorest resistance to natural weathering. No appreciable difference in overall durability was noted between the concretes made with cement Types II, IV, V, and the treated cements.

The concrete boxes prepared with cements Nos. 15, 18, 31, and 33 had the poorest durability. The condition for all four rows of the box specimens prepared with these cements is shown in figure 8. These four cements had C_3A contents of more than 10 percent. Conversely, the best overall ratings were given the concrete boxes prepared with cements Nos. 24, 43, 43A, 12T, 16T, and 21T. With the exception of No. 12T, all of these cements had a low C_3A content. In general, the concrete boxes made with the cements having a high C_3A content had the poorest durability. The exceptions were the concrete boxes prepared with cements Nos. 12, 17, and 12T. The durability of the concrete boxes prepared with cements Nos. 12 and 17 was poor only for row 3.

Fifteen of the 24 box specimens made with treated cement had better (lower) ratings than those prepared with the corresponding untreated cement from the same plant, 6 had equal ratings, and only 3 had poorer ratings. The treated cements showed to best advantage in row 4 specimens, which contained low-cement, high-slump concrete. The poorest durability for the concrete containing the treated cements was for the box specimens containing cements Nos. 11T, 18T, and 33T, each of which had a high C_3A content. The concrete made with cement No. 12T, which also had a high C_3A content, had good durability.

The air content for the concretes prepared with the treated cements Nos. 11T, 16T, and 33T were high; for the concretes prepared with treated cement No. 21T, it was moderate; and for cements Nos. 12T and 18T, it was low. For the conditions of exposure, the durability of the concretes made with the treated cements appeared to have been influenced by the chemical composition of the cement as well as the air content of the concrete.

Results of Tests on Beam Specimens

The three beams for each mix, made from the same batches of concrete as the boxes, were placed in the exposure plot and moist cured for 28 days, and were examined at the same time as the boxes. Determinations of length and sonic modulus were made. The results of these determinations were compared with results of similar determinations made just before the specimens were placed in the exposure plot. To minimize the effects of moisture and temperature, the beams were brought into the laboratory and stored in water, at a temperature of 73° F., for 5 to 7 days prior to the periodic determinations.

Table 8 data show the changes in the sonic moduli (N^2) that developed in the beams from the time of the initial readings until the time of the 1961 examination. The data indicate only a very general correlation between the loss of N^2 for the beams and the condition rating that was given comparable boxes. Decreases and increases in N^2 were about evenly divided among the beams made from the same batch of concrete as the 38 boxes that were assigned a rating of more than 3 in the 1961 examination. However, a decrease in N^2 in the beams was noted in 60

Table 9.—Change in length of beams after 12 to 16 years of exposure[1]

Cement, type and number	Percent increase or decrease in length, in rows—			
	1	2	3	4
I:				
11	0.140	0.068	0.099	0.089
12	.074	.028	.026	.032
13	−.001	−.006	−.021	−.001
14	.054	.009	−.002	−.006
15	.065	.008	.008	[2].208
16	.011	−.004	−.014	−.004
17	.011	.007	−.021	.008
18	.049	.021	.032	[2].369
Average	.050	.016	.013	.087
II:				
21	−0.009	−0.005	−0.024	−0.002
22	.004	.006	−.012	.022
23	−.009	−.012	−.086	−.006
24	.020	−.005	−.018	.004
25	.013	−.001	−.025	.006
Average	.004	−.003	−.033	.005
III:				
31	0.133	0.081	0.032	[2]0.259
33	.101	.063	.062	[2].163
34	.036	.019	−.023	[2].187
Average	.090	.054	.024	.203
IV:				
41	0.001	−0.002	−0.022	0.007
42	−.011	−.014	−.033	.009
43	−.003	−.013	−.029	.004
43A	.001	−.017	−.029	−.004
Average	−.003	−.012	−.028	.004
V:				
51	−0.008	−0.015	−0.029	0.026
Treated:				
11T	0.069	0.039	0.010	0.024
12T	.029	.011	.066	.030
16T	−.008	−.009	−.027	−.017
18T	.034	.001	−.002	.022
21T	−.008	−.010	−.069	−.002
33T	.029	.002	−.007	.010
Average	.024	.006	−.015	.011

[1] Each percent is the average of tests on three 3- x 4- x 16-inch beams.
[2] Beams showed numerous cracks.

percent of all of the specimens made with Types I and III cements. The boxes prepared with these two types of cement had the poorest overall ratings. An increase in N^2 was noted in more than 95 percent of the beams prepared with Types II, IV, and V and in 60 percent of those prepared with the treated cements.

The groups of beams prepared with the lean mix (4½ bags per cubic yard) and cement Types I and III showed the greatest loss in N^2. The five sets of beams prepared with cements Nos. 15, 18, 31, 33, and 34, had an average loss in N^2 of more than 30 percent at the time of the 1961 examination. All of the cements, except No. 34, had high C_3A contents. Numerous cracks had occurred in these beams. Their corresponding boxes were rated either 9 or 10 and were considered to be disintegrated. However, five sets of the lean-mix beams showed an average increase in N^2 of more than 20 percent. These beams were made with cements Nos. 21, 41, 42, 43A, and 21T. The corresponding boxes were rated 5, 4, 5, 3, and 1, respectively.

The lack of correlation between the two methods of determining resistance to weathering is believed to be due, in part, to (1) differences in the strength-gaining characteristics of concretes made with the different cements and (2) differences in moisture conditions caused by the basins of the boxes being kept full of water. Because of the latter conditions, large percentages of the concrete in the box specimens were sound, except for scaling of the floors and the slopes of the basins and radial cracks in the bottoms.

Table 9 lists the changes in length of the beams between initial readings and the 1961 examination. An analysis of this data indicates only a very general relation between the changes in length of the beams and the condition rating of comparable boxes. The results are similar to those obtained from tests for changes in sonic moduli. The beams prepared with the Types I and III cements had the greatest increases in length. The beams prepared with the Types II, IV, V, and the treated cements had little or no increase in length. The five sets of beams that had a loss in N^2 of more than 30 percent also had expansion of more than 0.15 percent. The same reasons for lack of correlation between the loss in N^2 of the beams and the condition rating of the boxes apply to length change.

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139 Million Drivers in 1980

Reported by E. M. COPE, Chief, Highway Statistics Division, and ARLENE R. MUNDY, Economist Office of Planning

BY THE HIGHWAY STATISTICS DIVISION BUREAU OF PUBLIC ROADS

Knowledge of the numbers, ages, and sex of licensed drivers is useful to public agencies in planning, research, making administrative decisions, and in formulating and carrying out safety programs. It is also useful to both Government and industry in forecasting, and to the latter in market research. Although requirements for obtaining driver licenses vary among the States, and the statistics concerning the numbers of drivers' licenses vary in completeness and significance, the information now available can be expanded to useful totals. Some of the States know how many drivers they have licensed; a few have made summaries by sex or age groups. Selected data from the States are presented, and on the basis of these and other information, the numbers of male and female drivers licensed are estimated separately for each State. Estimates of the sex and age grouping of licensed drivers are also given for the United States as a whole. It is forecast that the number of drivers' licenses will reach 125 million by 1975, and 139 million by 1980.

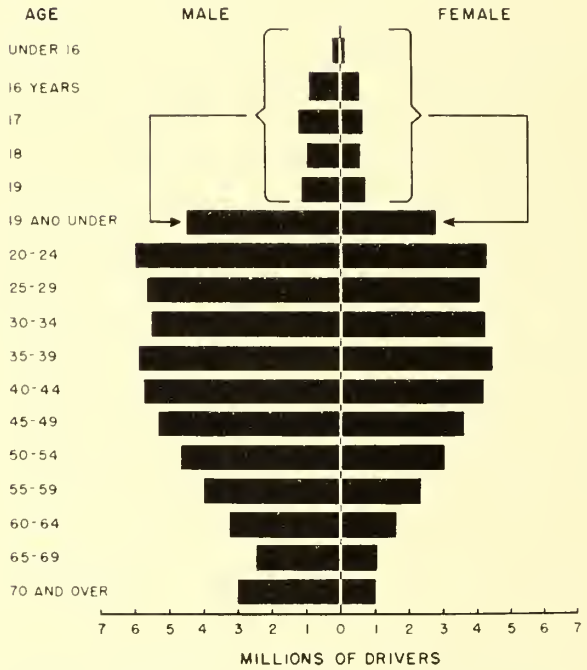

Figure 1.—Estimated number of licensed drivers, by age and sex, 1963.

139 Million Drivers in 1980

JUST 16 YEARS from now, in 1980, there will be 139 million licensed drivers of vehicles in the United States. There are already, in 1964, about 95 million licenses held by the persons who drive the approximately 85 million automobiles, trucks, and buses now registered. These drivers are the object of an almost endless barrage of advice, suggestions, admonitions, and beckonings, but surprisingly little is known about them.

Although all States require vital information, such as age and sex, to be stated on the application for a driver's license, only a few States have recognized and extracted these data as valuable aids in analyses of accident exposure and rates, the formulation and direction of highway safety programs, or the many other uses to which such data could be applied. Where it is available, data on the sex and age distribution of vehicle drivers is also a useful source of information for social and economic

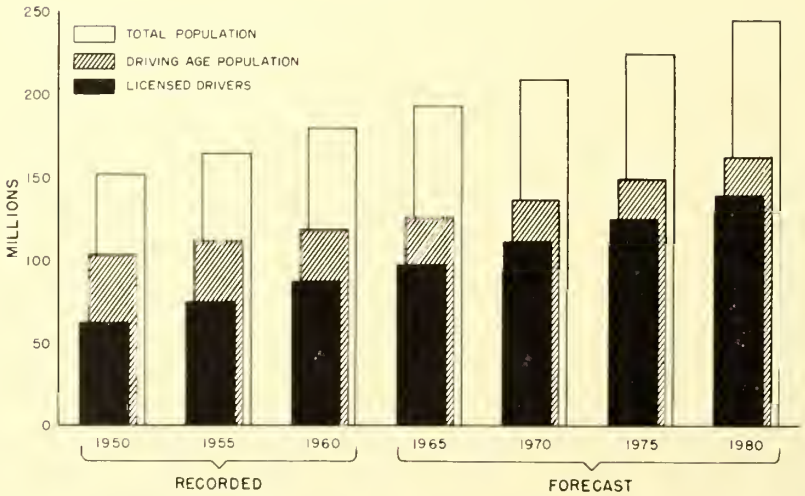

Figure 2.—Comparison of numbers of licensed drivers with total population and driving age population, 1950–80.

Table 1.—Administration and terms of state operators' and chauffeurs' licenses, Jan. 1, 1964

State	Office of issue	Learner's permit		Operator's license					Chauffeur's license		
						Minimum age					
						Unlimited operation except		Restricted operation for minors			
		Length of term [1]	Minimum age [2]	Length of term (years)	Renewal date	[3]	[4]		Length of term (years)	Renewal date	Minimum age [2]
Alahama	Driver License Division, Dept. of Public Safety.	1 month	15	2	Birthday	16			14		
Alaska	Dept. of Public Safety	60 days	14	3	Birthday	18	*16		[5] 13	[6] 1	Sept. 2	18
Arizona	Motor Vehicle Div., Highway Dept.	5 months*	15 yr., 7 mo.	3	Birthday	18	*16			2	Birthday	18–21
Arkansas	Motor Vehicle Div., Revenue Dept.	60 days	14	1	Jan. 1	18	*14		13	1	Jan. 1	18–21
California	Div. of Drivers Licenses, Dept. of Motor Vehicles.	6 months	15½	3 and 5	Birthday	16		14		(6)		
Colorado	Motor Vehicle Div., Dept. of Revenue.	5 months	15½–16	3	Birthday	17	16		[5] 14	3	Birthday	17
Counecticut	Div. of Registry, Dept. of Motor Vehicles.			2	Birth month	21	16			1	May 1	21
Delaware	Motor Vehicle Dept., Highway Commission.	60 days	16	[7] 2	Birthday	16				[7] 2	Birthday	18
Florida	Drivers License Div., Dept. of Public Safety.	2 years	14	2	Birth month	18	*16	*14		2	Birth month	18
Georgia	Drivers License Div., Dept. of Puhlic Safety.	1 year	15	1 or 5	Birthday	16				1 or 5	Birthday	18
Hawaii	County Police Dept	60 days*	15	Indefinite		20	*15		15	1	Issuanee	18–21
Idaho	Motor Vehicle Div., Dept. of Law Enforcement.	120 days	16	2	Birthday	16		14		1	Birthday	18
Illinois	Driver License and Safety Responsibility Div., Motor Vehicle Dept.	6 months	15–16	3	Birthday	18	*16	*15		1	Issuance	18–21
Indiana	Drivers Licensing, Bur. of Motor Vehicles.	1 year*	15–16	2	Birth month	18	*16	*15		1	Birth month	18–21
Iowa	Safety Responsibility Div., Dept. of Public Safety.	2 years	14	2	Birthday	16		14		1	Birthday	18
Kansas	Driver License Div., Motor Vehicle Dept.	2 months	14	2	Birthday	16		14	14	2	Birthday	16–18
Kentucky	Div. of Driver Licensing, Dept. of Public Safety.	2 months	16	2	Birth month	18	*16			1	Jan. 1	18–21
Louisiana	Driver License Div., Dept. of Public Safety.	2 years	15	2	Birthday	15			14	1	Issuanee	18
Maine	Motor Vehicle Dept., Dept. of State.	1 year	15–17	2	Birthday	18	*17	*15	17			
Maryland	Dept. of Motor Vehicles	60 days*	16	2	Birth month	21	*16		16	2	Issuance	16
Massachusetts	Registry of Motor Vehicles	6 months	16–18	2	Birthday	18		*16		[8] 1	Issuance	21
Michigan	Div. of Driver and Vehicle Services, Dept. of State.	60 days	16	3	Birthday	18	*16	*14	15	1	Issuance	18
Minnesota	Drivers License Div., Dept. of Highways.	6 mouths	15	4	Birthday	21	*16		[5] 15	1	Birthday	18
Mississippi	Drivers License Div., Dept. of Public Safety.	60 days	15	1 or 2	Issuance	15				1 or 2	Issuance	17
Missouri	Driver's License Unit, Dept. of Revenue.	60 days	15–16	3	Issuance	16				1	Issuance	18–21
Montana	Highway Patrol	6 months*	13–15	2	Birthday	21	*15	*13		2	Birthday	18–21
Nebraska	Driver's License and Safety Equipment, Dept. of Motor Vehicles.	1 year	15	2	Sept. 1 odd year.	20	16					
Nevada	Drivers License Div., Dept. of Motor Vehicles.	6 months	15½	[9] 5	Birthday	18	*16	14	[5] 14	5	Birthday	18
New Hampshire	Div. of Motor Vehicles, Dept of Safety.	School term.	15–16	2	Birthday	16		15	16	[10] 2	Birthday	18
New Jersey	Licensing Service, Motor Vehicle Div., Dept. of Law and Puhlic Safety.	2 months*	17	1 or 3	Issuauce	17		16	17	(11)	Indefinite	21
New Mexico	Drivers Service Div., Dept. of Motor Vehicles.	2 months	14–16	2	Birth month	18	16	15	13	1	Birth month	18
New York	Dept. of Motor Vehicles	6 months	16	3	Issuance	18		16		3	Issuance	18
North Carolina	Driver License Div., Dept. of Motor Vehicles.	1 month	16	4	Birthday	18	*16			2	Birthday	18–21
North Dakota	Safety Responsibility Div., Highway Dept.	3 months	13	2	Birth month	16		*13				
Ohio	Driver License Sect., Ohio State Highway Patrol.	6 mouths*	16	3	Birthday	21	16	14	16	3	Birthday	18
Oklahoma	Driver License Div., Dept. of Public Safety.	6 months	15½–16	2	Birth month	16			14	2	Birth month	18–21
Oregon	Drivers License Div., Dept. of Motor Vehicles.	1 year	15	2	Birthday	16		14		2	Birthday	18
Pennsylvania	Bur. of Motor Vehicles, Dept. of Revenue.	3 or 4 months.*	16	2	Feb. 1	18		*16				
Rhode Island	License Div., Registry of Motor Vehicles.	90 days	16	2	Oct. 1	16				2	Oct. 1	18–21
South Carolina	Motor Vehicle Div., Highway Dept.	6 months	14	4	July 1	16		14		1	Jan. 1	18
South Dakota	Dept. of Motor Vehicles	60 days	14	4	Birthday	16		14				
Tennessee	Driver License Div., Dept. of Safety.	10 days	16	2	Birthday	16			14	2	Birthday	18–21
Texas	Dept. of Puhlic Safety	2 years	14–16	2	Issuance	18	*16	14	14	1	Issuauce	16–17–21
Utah	Drivers' License Div., Dept. of Public Safety.	4 months	15½–16	3 and 5	Birthday	16	15½			3 and 5	Birthday	18–21
Vermont	Motor Vehicle Dept	1 year	15	1	Birthday	18	16					
Virginia	Div. of Motor Vehicles	90 days	15	3	Birth month	18	*15			1	Issuance	18–21
Washington	Operator's Lieense Div., Dep. of Licenses.	6 months	15	2	Birthday	21	*16					
West Virginia	Opertrs. & Chauff. Div. and Driver Imprvmt. Div., Dept. of Motor Vehicles.	2 months	16–21	4	Issuance	16			16	1	Issuance	18–21
Wisconsin	Motor Vehicle Dept	6 months	16	2	Birthday	16	14			1	Birthday	18
Wyoming	Motor Vehicle Div., Dept. of Revenue of the Board of Equalization.	90 days	15	3	Birthday	21	*15		14	1	Issuance	18–21
Dist. of Col	Dept. of Motor Vehicles	60 days*	16	3	Issuance	18	*16					

[1] An asterisk (*) is shown if the learner's permit is mandatory before obtaining an operator's license.

[2] Where more than one age is shown for a State, the lower age (or ages) is qualified by certain restrictions, such as completion of driver training courses, times of operation, type of operation, proof of necessity in hardship cases, and parent or guardian approval.

[3] No special provisions for miuors. [4] Special provisions for minors and an asterisk indicates that signature of parent or guardian is mandatory before issuance of license.

[5] Motor scooter license only.

[6] Drivers' licenses are not designated operators' or chauffeurs' licenses. The applicant is required to take an examination appropriate to the type of motor vehicle he will operate, class 1, 2, or 3.

[7] Indefinite issue for drivers meeting specified requiremeuts. Taxicab drivers' licenses are issued only to persons 21 or over, are issued annually, and expire May 31.

[8] Required only for school bus operators.

[9] Two years for those over 65 years old.

[10] Commercial license is required for any person operating a vehicle having more than 1-ton capacity and not owned by the operator.

[11] For-hire hus operator's license is issued for an indefinite period, but evidence of physical fitness, good character, and experienee must he furnished each 12 months.

Table 2.—Number of States issuing drivers' licenses at specified minimum ages, 1964

Minimum age of issuance	Number of States issuing—		
	Learner permits	Junior or special licenses (restricted operation)	Regular driver licenses (unlimited operation)
13	2	2	----
14	9	12	2
15	23	5	7
16	15	4	37
17	1	----	2
18	----	----	3
TOTAL	50	23	51

Table 3.—Specified periods for which States issue learners' permits and regular drivers' licenses, 1964

Period for which issued	Number of States issuing—	
	Learner's permits	Regular driver's licenses
Less than 3 months	18	----
3–6 months	21	----
School term	1	----
1 year	6	2
2 years	4	27
3 years	----	12
4 years	----	5
5 years	----	4
Indefinite	----	1
TOTAL	50	51

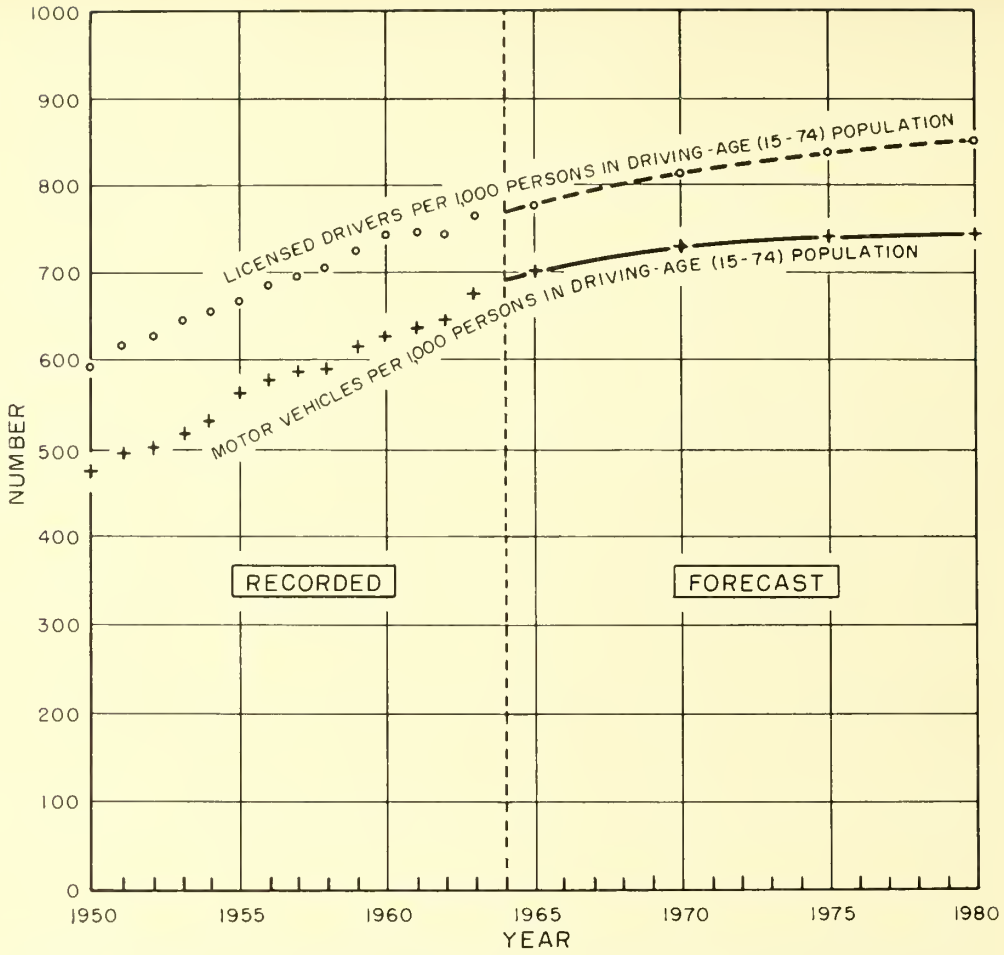

Figure 3.—Relationship of the numbers of licensed drivers and vehicle registrations to population in the vehicle-operating age groups.

analysts, and for business in the analysis of wants and needs, forecasting of markets, and efficiency in distribution and advertising. Fortunately, there is an increasing recognition of the potential value of the information that the licenses can produce, in addition to their primary function of certifying the ability of the holder to operate a vehicle, and maintaining a record of his traffic law violations.

From a highway safety standpoint, alone, the nearly universal ownership and operation of vehicles require more attention to the qualifications and practices of individual drivers. This has been evidenced by several major developments in recent years. The list is impressive. Without attempting to put them in their order of importance, it includes highly organized high school driver education programs, the point system for control of hazardous drivers, revamping and tightening of driver licensing by the States, more adequate staffing and record systems, and the establishment, by the Federal Government, of the Federal Driver Register, in which all of the States cooperate in exchanging information on individual drivers having records of offenses leading to suspension or revocation of their licenses. The District of Columbia is treated as a State for the purposes of this study. It should be remembered, when making comparisons, that it is a compact central city having a highly developed public transportation system, and that its statistics exclude the suburban and rural areas that tend to balance or average the statistics of the 50 States. The statistics on driver licenses, vehicle ownership, and related factors for other major cities, if obtained separately and not submerged in the totals of their States, would undoubtedly be similar to those of the District of Columbia.

Driver license administration, requirements, and fees

Generally, use of the information from a driver's license is aided by knowledge of the minimum qualifications required to obtain the license, the ease with which it can be obtained, the length of time it is valid, and its cost. Any of these factors, and perhaps others as well, can affect the significance of the data.

These factors, and other significant information, are shown for each State in tables 1, 4, 5, and 6. Table 1 lists the agency in each State that administers the issuance and recording of licenses. It also lists the minimum ages at which licenses may be obtained, the lengths of the terms for which they are issued, and the factors governing the dates of expiration and renewal. All States except Connecticut issue learners' permits. Forty-one States issue junior or other restricted licenses. The restrictions differ among the States and are too numerous to be listed. Such licenses, generally, are issued to persons too young to qualify for regular licenses and are to permit operation of a vehicle to and from school, for shopping or marketing, or for making other trips under prescribed conditions (see table 2). The restrictions may also cover the type of vehicle to be operated, operation between sunrise and sunset, and other factors, such as insurance coverage requirements or requirements for written approval of parents.

The terms of both learner permits and regular driver licenses, as may be seen in table 1, differ considerably among the States. In some, learner permits are issued for a period as short as 1 month, but four States issue them for periods as long as 2 years. The great majority are for 6 months or less.

The driver licenses of more than half of the States are issued for 2-year terms. An even dozen issue them for 3 years, and only two for 1-year terms. Although issuance of licenses for indefinite terms was at one time fairly common among the States, Hawaii is now the only one in which an indefinite term regular driver's license may be obtained. For certain drivers with safe driving records, Delaware issues an indefinite term license, for which the fee is $10. Relatively few of these are issued, and for purposes of this study, they are considered to be special licenses. A summary of the terms of State drivers' licenses is shown in table 3.

Table 4.—Driver license examination, renewal, and reciprocity, Jan. 1, 1964

State	Kind of examination required for original or renewal of drivers' license [1]					Age requirement for reexamination for renewals	Renewal notices mailed [2]		Expiration date extended for military personnel during service and after discharge	Reciprocity				Point system in use [3]
										New residents			Nonresidents	
	Written	Oral	Driving	Vision	Other		Operators	Chauffeurs		Time limit to obtain driver license after establishing residence	Must surrender license from former State	Examination for persons having legal license from former State	May obtain license	
Ala	F		F	F			Yes*		No	1 month	Yes	Driving	No	Yes.
Alaska	F		F	F & R	Sign recognition	70	No	No	No	90 days	No	None	No	No.
Ariz	F		F	F & R		65	Yes	Yes	No	Immediately	No	Entire	No	No.
Ark	F	F	F	F	As necessary		No	No	No	Immediately	Yes	No	No	No.
Calif	F & R	F & R	F	F & R	Hearing, signs, simple English.		No	No	Duration of service.	10 days	Yes	Entire	Have Calif. address.	Yes.
Colo	[4] F & R	If illiterate	[4] F & R	F & R	Physical, aptitude.		No	No	[5]	30 days	Yes	Driving waived.	No	Yes.
Conn	F	If illiterate	F	F	Sign recognition.		Yes [6]	Yes [6]	No	60 days	Ill. and S.D. only.	Driving waived.	Valid in Conn. only.	Yes.
Del	F		F	F		63	Yes [6]	Yes [6]	90 days	90 days	Yes	Written and vision.	No	No.
Fla	F	F	F	F	Signs, veh. insp.		No	No	Duration of service.	Immediately	Yes	Entire	Pass entire exam.	Yes.
Ga	F	F	F	F	Sign recognition.		No	No	No	30 days	Yes	Written and vision.	No	No.
Hawaii	F		F	F						90 days	No	None		Yes.
Idaho	F	F	F	F & R			No	No	No	Immediately	Yes	Entire	No	Yes.
Ill	F	F	F	F	Signs and laws	69	No	No	No	90 days	Yes	Entire	Chauf. employed in Ill.	Yes.
Ind	[7] F & R	If illiterate	F	[8] F		75	No	No	Duration of service.	Indefinite	Yes	Written and vision.	No	Yes.
Iowa	F	If illiterate	F	F & R			No	No	6 months	Immediately	No	Entire	No	No.*
Kans	F		F	F			Yes*	Yes	30 days	90 days	Yes	Entire	Servicemen only.	Yes.
Ky	F	If illiterate	F	F			No	No	No	Immediately	Yes	Entire	No	Yes.
La	F	F	F	F & R			No	No	60 days	90 days	Yes	Entire	No	Yes.
Maine	F		F	F		[9] 75	Yes		30 days	Immediately	Yes	Driving waived.	Pass entire exam.	Yes.
Md	F		F	F	Sign recognition.	60	Yes	No	No	30 days	Yes	Driving waived.	Chauf. employed in Md.	Yes.
Mass	F		F	F			Yes [6]		60 days	Immediately	No	Driving waived.	Upon application.	No.
Mich	F	F	F	F & R	Physical		No	No	No	Immediately	No	Driving waived.	No	Yes.
Minn	F		F	F	Doctors, report		No	Yes	90 days	60 days	Yes	Entire	No	Yes.
Miss	F	If illiterate	F	F	Sign recognition.		No	No	No	60 days	Yes	Entire	No	No.
Mo	F	If illiterate	F	F			Yes*	Yes	No	Immediately	Yes	Entire	Chauf. employed in Mo.	Yes.
Mont	F	If illiterate	F	[8] F			Yes*	Yes	30 days	90 days	Yes	Entire	No	Yes.
Nebr	F		F	F			No	No	No	30 days	No	Entire	No	Yes.
Nev	F		F	F & R			No	No	Duration of service.		Yes	None	No	No.*
N.H	F	If illiterate or non-English.	F	F		75	Yes [6]	Yes [6]	No	[10]	Yes	Driving waived.	Valid in N.H. only.	No.
N.J	F		F	F			Yes*		90 days	60 days	No	Driving waived.	Valid in N.J. only.	Yes.
N.Mex	F		F	F			No	No	No		Yes	Entire	90-day permits	No.
N.Y	F	F	F	F	Medical	[11] 66	Yes	Yes	[12]	30 days	No	Not if reciprocity agreement exists.	Valid in N.Y. only.	Yes.
N.C	F & R	F & R	F & R	F& R			Yes	Yes	No	Immediately	Yes	Driving waived.	No	Yes.
N.Dak	F	F	F	F			Yes*		No	60 days	Yes	Entire	No	No.
Ohio	F	F	F	F			No	No	No	Immediately	Yes	Entire	No	Yes.
Okla	F	F	F	F			Yes	Yes	No	Immediately	Yes	Entire	By surrendering home State license.	Yes.
Oreg	F	F	F	F	Sign recognition		Yes*	Yes	No	Immediately	Yes	Entire	No	No.
Pa		F	F	F	Physical		Yes		Duration of service.	30 days	No	Driving waived.	No restriction	No.
R.I	F		F	F		70	Yes*	Yes	30 days	Immediately	Yes	Entire	If employed in R.I.	No.
S.C	F	F	F	F			No	Yes	90 days	90 days	Yes	Entire	Chauf. only	Yes.
S.Dak	F		F	F & R			No		Duration of service.	90 days	Yes	Entire	Operators only.	No.
Tenn	F		F	F			Yes*	Yes	Duration of service.	Immediately	Yes	Entire	No	No.
Tex	F	If illiterate	F	F	Sign recognition		Yes*	Yes*		Immediately	No	Entire	If needed	No.
Utah	F		F	F & R			No	No	90 days	60 days	No	Entire	Upon application.	Yes.
Vt	F	F	F	F			Yes*	No	[13]	6 months	No	Driving waived.	Valid in Vt. only.	No.
Va	F	F	F	F			Yes	No	6 months	Immediately	No	Driving waived.	Pass entire exam.	No.
Wash	F		F	F & R			Yes*		No	30 days	Yes	Entire	No	Yes.
W.Va	F		F	F			Yes*	Yes	6 months	30 days	Yes	None	No	Yes.
Wis	F		F	F	Sign recognition		Yes*	Yes	No	Immediately	Yes	Entire	No	Yes.
Wyo	F		F	F			No	No	No	90 days	No	Dept. may waive exam.	No	No.
D.C	F	If illiterate	F	F	Reaction	65	Yes*		Up to 6 years.	[10]	No	Driving waived.	Valid in D.C. only unless a diplomat.	Yes.

[1] F=original license; R=renewal. Other examinations apply to the original issue, or in special cases, or as needed for renewals.
[2] Asterisk indicates renewal notice can be forwarded if addressee has moved.
[3] Asterisk indicates law provides for point system but is not yet in use.
[4] Renewal applicants are required to take written and driving tests if they have had a traffic violation during the term of license, or at the discretion of the examiner.
[5] For civilians, expiration date may be extended up to 2 years, depending upon need and circumstances; for military and dependents, up to 3 years, or 90 days after return to Colorado, whichever is sooner.
[6] Operators and chauffeurs renewal notices are mailed, except those under suspension or revocation.
[7] Written test required every 4 years.
[8] Vision test required for renewals every 4 years in Indiana, and for every third renewal in Montana.
[9] If the applicant has been examined within the past year, or has received a license effective on his 74th birthday, he is required to be examined at 76 years and every renewal thereafter.
[10] Ranges from immediately to 6 months depending on reciprocity with each State.
[11] Reexamination for those over 65 if involved in an accident or if requested by insurance company.
[12] For operators, expiration date may be extended to the Sept. 30 next succeeding 60 days after separation; for chauffeurs, to the next May 31 next succeeding 60 days after separation.
[13] Maximum of 4 years beyond original expiration date or 30 days after discharge from the Armed Forces, whichever occurs first.

Table 4 shows the general State provisions for driver license examinations, renewals, and reciprocity. All States require driving and vision tests for applicants who have not previously been licensed to drive, but many waive the driving test for persons holding valid licenses issued by other States. It is interesting to note, in table 4, that several States issue licenses to persons who cannot read. While some road signs have distinctive shape or color, others do not, and one cannot help wondering what mysteries road signs and warnings must hold for such drivers.

The requirement of a vision test at each driver's license renewal is a somewhat new development that is now in effect in 13 States. In addition, Indiana requires a vision test on every second renewal of a license, and Montana requires one on every third renewal. The 15 States requiring retesting of vision are pleased with the results, and it is probable that the requirement will be adopted by others. A ripple of surprise followed one State's discovery, recently, that some persons receiving aid for the blind were also holders of valid drivers' licenses—a situtation since corrected in that State.

Although the driver license laws of many States are sufficiently broad to permit the administrative requirement of reexamination of licensed drivers for cause, or under specific circumstances, at present only 11 have instituted programs of mandatory reexamination of applicants who have reached a specified age. The States requiring reexaminations and the ages at which they are required are shown. Some States mail notices of approaching license expiration and need for renewal. These account for a significant part of the administration cost of the States (table 4) that do send renewal notices.

Some States make special provisions for the extension of the terms of drivers' licenses of persons in the armed services (table 4). Where a specified time is indicated, it is the period for which the license remains valid after the termination of active service. Such extensions do not apply to service in the reserves or to temporary service in the National Guard.

Nearly all States require that a person residing in the State and employed therein must obtain a driver's license to qualify for the operation of vehicles within the State. Most, however, allow a period of grace. These are shown in table 4, but the definitions of residence and employment differ somewhat among the States, and should be checked locally if any question of individual compliance is involved.

Table 4 also reflects, in its last column, the fact that 30 of the States have adopted a point system of dealing with traffic law violators. In addition, the legislatures of two more States have authorized such systems, but have not yet put them into operation.

As may be seen in table 5, the information on driver licenses differs among States. The only items included in all are name, address, signature, and birth date.

The driver licenses of all States show date of expiration, except in Hawaii, where they are issued for an indefinite term. Surprisingly, the licenses of four States do not show weight, and those of four do not show sex. Included in these, Connecticut, Massachusetts, and Pennsylvania licenses show neither. The Pennsylvania license is the only one that does not show height. Never included on the driver licenses of all States, race remains an item shown on those of 29 States.

Although still appearing on the licenses of all but seven States, color of hair probably remains as a vestige of days when more confidence could be placed in it as an item of identification. Artificial hair coloring and even artificial hair have become so commonplace that color of hair has lost much of its value as an item of personal description. Some States have already dropped it from the license description, and others are considering dropping it. All States except Connecticut, Maryland, and Pennsylvania show color of eyes on their licenses.

A new development is the inclusion of a photograph as part of the driver's license, now a requirement in seven States. Colorado, the District of Columbia, Louisiana, and New Mexico use color photographs.

Although the items of information shown on the drivers licenses (table 5) may be dropped or added to from time to time, there is a remarkable degree of uniformity in the inclusion of major items required for identification, administration, and law enforcement. This uniformity, and the tendency to increase, of both the uniformity and the care with which it is administered, give added value and significance to the data available from the State records of licenses.

As may be seen in table 6, the fees for drivers' licenses are scarcely more than nominal. In many States they hardly cover the cost of administering the licenses. The fees have never been regarded by the States as a source of revenue beyond the amounts needed to support the function, and have remained low. One result of this is that licenses are readily obtainable by persons in all economic levels. Since car ownership is not required, and possession of a driver's license appears to be almost a social requirement of the modern American scene, the effort to qualify seems to be almost universal among young people. Thus, there is created a broad-based source of data, in driver license records, that is useful for many purposes. These data should contain little bias of an economic nature; and since a license, once obtained, is relinquished only with great reluctance, the data source is almost certain to improve with the passage of time.

The fee for a driver's license, divided by the term for which the license is issued, yields the annual cost. The highest annual cost, $4, is in Rhode Island, which collects $8 for a 2-year license and puts the revenue from this source in its general fund without earmarking it for highway purposes, as most States do. South Carolina's driver license, at 50 cents, is the lowest in original cost; and since it is for a 4-year term, its 12½-cent annual cost is also the lowest, with the possible exception of Hawaii's, which is issued for an indefinite term. The costs of driver licenses, shown in table 6, are summarized in table 7.

Availability of data from drivers license records

There are several reasons why the States do not extract more driver license information from their files, but the principal underlying cause probably is the tendency to look on the records as places where one can ascertain that an individual has met the requirements for a license, and as a record of information on individual drivers. This largely explains, too, the fact that many of the States cannot supply a count of total drivers licenses in force. But another important factor is the need for modern data processing equipment capable of handling the sheer magnitude of the job of extracting the data from the records. As the equipment and staff become available, the States are beginning to put driver license data on punch cards or tapes and to summarize their contents.

Age and sex distribution of licensed drivers

As of the summer of 1964, 19 States were able to supply counts of drivers' licenses outstanding at the end of 1963, segregated by age groups and by sex. These, listed in table 8, are undoubtedly indicative of the age and sex groupings of other States. In the 19 States for which 1963 data were available, 60.2 percent of the licensed drivers were male, and 39.8 percent were female; and in none of the 19 States did the ratio change as much as 3 percentage points from this average.

Perhaps the most noticeable characteristic of the drivers license distribution of the 19 States given in table 8 is the fact that males substantially outnumber females in all States and all age groups. Although differences between the age and sex groupings of licensed drivers among the States are significant, there is also a surprising degree of overall consistency. The highest percentage of male drivers, Kentucky's 62.8 was only 5.1 percentage points higher than Delaware's low of 57.7. Since the populations of Kentucky and Delaware are 49.6 percent and 49.5 percent male, respectively, differences in the elements of motivation and vehicle usage, rather than in the male-female ratios of the total population, must account for the difference between the two States in the percentages of male versus female drivers.

Despite the stability of the overall male-female ratios of the States, the State-to-State age group compositions of licensed drivers differ considerably. Probing for the reasons for these differences is beyond the scope of this study, but they undoubtedly could be found, and their significance measured. Some of the differences may be attributable to the economic characteristics of the States, or to the age and sex distributions of the populations. Wage levels, per capita income, urban-rural relationships, topography, climate, and other factors might be expected to influence the makeup of the driving population.

As expected, it was found that the ratios of male-to-female drivers increase substantially and progressively in the age groups 55 and above. This statistical fact takes on added

Table 5.—Items included on operator's license, Jan. 1, 1964

State	Name	Address	License number	Color of eyes	Color of hair	Photograph	Finger or thumb print	Restrictions (eye glasses, automatic transmission, etc.)	Expiration date	Race	Weight	Height	Signature of operator	Sex	Birth date	Violations resulting in convictions	Other items
Alabama	X	X	X	X	X			X	X	X	X	X	X	X	X		
Alaska	X	X	X	X	X			X	X		X	X	X	X	X		[1].
Arizona	X	X	X	X	X	X		X	X	X	X	X	X	X	X		
Arkansas	X	X	X	X	X				X	X	X	X	X	X	X	X	Date of issue.
California	X	X	X	X	X	X	Optional	X	X		X	X	X	X	X		
Colorado	X	X	X	X	X	X	Right index	X	X		X	X	X	X	X		
Connecticut	X	X	X				Pub. sve. opr. only.	X	X			X	X		X	X	
Delaware	X	X	X	X	X	Chauf. only		X	X	X	X	X	X	X	X		
Florida	X	X	X	X	X			X	X	X	X	X	X	X	X	X	
Georgia	X	X	X	X	X			X	X	X	X	X	X	X	X	X	
Hawaii	X	X	X	X	X	Optional	Right thumb	X	None	X	X	X	X	X	X	X	[2].
Idaho	X	X	X	X	X			X	X	X	X	X	X	X	X		
Illinois	X	X	X	X	X			X	X	X	X	X	X	X	X		
Indiana	X	X	X	X	X			X	X	X	X	X	X	X	X		
Iowa	X	X	X	X				X	X	X	X	X	X	X	X	X	Soc. Sec. No.
Kansas	X	X	X	X	X			X	X	X	X	X	X	X	X		
Kentucky	X	X	X	X	X			X	X	X	X	X	X	X	X		
Louisiana	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X	
Maine	X	X		X	Optional			X	X		X	X	X	X	X	X	
Maryland	X	X	X					X	X	X	X	X	X	X	X		
Massachusetts	X	X	X	X	X			X	X			X	X		X		
Michigan	X	X	X	X	X		Chauf. only	X	X		X	X	X	X	X	X	Point system.
Minnesota	X	X	X	X	X			X	X		X	X	X		X		
Mississippi	X	X	X	X	X			X	X	X	X	X	X	X	X		
Missouri	X	X	X	X	X			X	X		X	X	X	X	X	X	
Montana	X	X	[3] X	X	X	X		X	X		X	X	X	X	X		
Nebraska	X	X	X	X	X			X	X	X	X	X	X	X	X		
Nevada	X	X	X	X	X		X	X	X		X	X	X	X	X		
New Hampshire	X	X	X	X	X			X	X		X	X	X	X	X	X	Blood type.
New Jersey	X	X	X	X	X			X	X		X	X	X	X	X		
New Mexico	X	X	X	X	X	X		X	X		X	X	X	X	X		[4].
New York	X	X	X	X		Chauf. only		X	X			X	X	X	X	X	
North Carolina	X	X	X	X	X			X	X	X	X	X	X	X	X		
North Dakota	X	X	X	X	X			X	X	X	X	X	X	X	X	X	[5].
Ohio	X	X	X	X	X	Chauf. only		X	X		X	X	X	X	X	X	Point system.
Oklahoma	X	X	X	X	X			X	X	X	X	X	X	X	X		
Oregon	X	X	X	X	X			X	X		X	X	X	X	X		
Pennsylvania	X	X	X					X	X				X		X		
Rhode Island	X	X	X	X	X			X	X		X	X	X	X	X		
South Carolina	X	X	X	X	X			X	X	X	X	X	X	X	X		
South Dakota	X	X	X	X	X			X	X	X	X	X	X	X	X		
Tennessee	X	X	X	X	X			X	X	X	X	X	X	X	X	X	
Texas	X	X	X	X	X			X	X	X	X	X	X	X	X		
Utah	X	X	X	X	X	Chauf. only		X	X	X	X	X	X	X	X		
Vermont	X	X	X	X	X			X	X		X	X	X	X	X		
Virginia	X	X	X	X	X			X	X	X	X	X	X	X	X		
Washington	X	X	X	X	X			X	X		X	X	X	X	X		
West Virginia	X	X	X	X	X			X	X	X	X	X	X	X	X		Blood type.
Wisconsin	X	X	X	X	X			X	X	X	X	X	X	X	X		
Wyoming	X	X	X	X	X			X	X	X	X	X	X	X	X		
Dist. of Col.	X	X	X	X	X	X		X	X		X	X	X	X	X		

[1] Issuance date includes code number for place of issuance.
[2] Date of issue and signature of examiner are also shown on the license. Violations resulting in convictions and points under the point system are entered by the court.
[3] Applicant's birth date is included in the license number; first letter of last name and year, day of birth, and month.
[4] Blood type, and physical and mental condition.
[5] Written warning of traffic violation, explanation of change of address notice, mailing of notice of renewal application, and basis for suspension and revocation.

Table 6.—Driver license fees and service charges, Jan. 1, 1964

State	Payment at time license is issued										
	Learner's permit fee		Operator's license fee			Chauffeur's license fee			Service charge [1]		
	Amount	Amount applied to operator's license	Original	Renewal	Duplicate	Original	Renewal	Duplicate	Learners	Operators	Chauffeurs
Alabama	[2] $0.35		$4.25	$4.25	$0.25				$0.10	$0.10	$0.10
Alaska	1.00		5.00	5.00	1.00	$2.00					
Arizoua	2.00		2.50	2.50	1.00	2.50	$2.50	$1.00			
Arkansas	No fee		2.00	2.00	2.00	5.00	5.00	1.00			
California	3.00	$3.00	3.00	3.00	1.00						
Colorado	2.25	2.25	2.25	2.25	1.25	5.25	5.25	1.25			
Connecticut			([3])	6.00	1.00	3.00	3.00	1.00			
Delaware	4.00	4.00	4.00	4.00	1.00	[4] 4.00	4.00	1.00			
Florida	3.00		3.00	3.00	.25	5.00	5.00	.25	[5] .50	[5] .50	[5] .50
Georgia	1.00		1.00 or 5.00	1.00 or 5.00	1.00	2.00 or 10.00	2.00 or 10.00	1.00			
Hawaii	1.00		3.00		.50	3.00	1.00	.50			
Idaho	3.00		4.00	4.00	.75	3.00	3.00	.75			
Illinois	3.00	3.00	3.00	3.00	1.00	5.00	3.00	1.00			
Indiana	1.00		1.50	1.50	1.50	1.50	1.50	1.50			
Iowa	3.00		3.00	3.00	.25	4.00	4.00	.50			
Kansas	.50		([6])	2.00	.50	([6])	4.00	.50			
Kentucky	1.00		2.00	2.00	1.00	2.00	2.00	2.00	1.00	.75	.25
Louisiana	2.50	2.50	2.50	2.50	1.50	[7] 3.50	[7] 3.50	1.50			
Maine	3.00		5.00	5.00	.50						
Maryland	5.00	5.00	7.00	2.00	.50	8.00	3.00	.50		.25	
Massachusetts	2.00		[8] 8.00	5.00	1.50	[9] 5.50	2.50	1.50			
Michigan	No fee		4.00	2.50	1.00	4.00	2.75	1.00		([10])	([10])
Minnesota	1.00		3.00	3.00	.50	3.00	2.00	1.00			
Mississippi	.50		2.50	2.50	1.00	4.50	4.50	1.00		[11] .25	[11] .25
Missouri	.25		1.00	1.00	.25	3.00	3.00	.25			
Montana	4.00	4.00	4.00	4.00	.50	4.00	4.00	.50			
Nebraska	1.00		3.00	3.00	.50						
Nevada	3.00	3.00	3.00	3.00	1.00	5.00	5.00	1.00			
New Hampshire			5.00	4.00	1.00	7.00	4.00	1.00			
New Jersey	2.00		3.00 or 8.00	3.00 or 8.00	2.00	No fee					
New Mexico	.50		3.25	3.25	1.00	2.75	2.75	1.00			
New York	.50		[8] 5.00	3.00	3.00	[8] 8.00	6.00	3.00	.50	.20	.20
North Carolina	No fee		2.50	2.50	.50	4.00	4.00	.50			
North Dakota	[12] 3.00	[12] 3.00	[12] 3.00	3.00	1.00						
Ohio	.75		1.00	1.00	.50	1.50	1.50	1.00	.25	.25	.25
Oklahoma	No fee		4.00	4.00	1.00	[13] 8.00	[13] 8.00	1.00	.20	.20	.20
Oregon	.50		2.75	2.75	.25	2.00	2.00	.25			
Pennsylvania	4.00	4.00	4.00	4.00	.50						
Rhode Island	No fee		[8] 13.00	8.00	1.00	[8] 13.00	8.00	1.00			
South Carolina	No fee		.50	.50	.50	2.00	2.00	No fee			
South Dakota	2.00		2.00	2.00	2.00						
Tennessee	4.00	4.00	4.00	4.00	2.00	6.00	6.00	2.00			
Texas	[14] 3.00	Total fee	3.00	3.00	.25	[15] 6.00	6.00	.25			
Utah	3.00	3.00	3.00	2.00	1.00	3.00	2.00	1.00			
Vermont	1.00		[8] 4.50	2.50	.50						
Virginia	No fee		2.00	2.00	.25	3.00	3.00	.25			
Washington	1.50	.50	[8] 6.00	4.00	.50						
West Virginia	4.00		5.00	5.00	1.00	3.00	3.00	1.00			
Wisconsin	1.50		[8] 4.50	2.00	1.00	[8] 5.00	2.00	1.00			
Wyoming	No fee		2.00	2.00	1.00	2.00	2.00	1.00			
District of Columbia	2.00		3.00	3.00	.50						

[1] All service charges are a part of the regular fees as listed, except Maryland and Mississippi which are in addition to the given fee.
[2] Learner's permit fee for 15-year-olds is $0.50.
[3] $5 examination fee plus $0.25 per month from date of issue to last day of month of next birthday and $3 or $6, depending upon year of birth.
[4] $0.50 on exchange from a valid operator's license.
[5] The county judge receives $0.50 of the stated fee for each of the first 10,000 licenses issued, and $0.35 each thereafter. $1 of each fee goes to the Driver Education Fund, and the balance to General Revenue.
[6] Operators, $1 to $3; chauffeurs, $2 to $6, depending upon year of birth, even or odd year. $3, additional if examination is needed.
[7] New Orleans Parish, $5.50.
[8] The difference between new and renewal license fee is the charge for examination when one is required, except in Wisconsin where the examination fee is $2.
[9] Required only for schoolbus operators.
[10] $2 for each original license and $0.50 for each renewal.
[11] The additional $0.25 is charged only when renewed by sheriff's office or renewal agents.
[12] $1.50 for 17 years and under. No charge is made for the learner's permit, but the license fee is collected at the time the permit is issued.
[13] Commercial chauffeur, $10 original, $10 renewal.
[14] Same as original license for which application is being made; $3 for operator, $4.50 for private chauffeur, $6 for commercial chauffeur.
[15] Commercial operators to transport property, $4.50.

significance when it is remembered that the life expectancy of women is now 73½ years, 6½ years longer than that of men in the United States. In the age groups in which the ratio of males to females is decreasing, the ratio of male-to-female licensed drivers increases substantially and progressively. This undoubtedly indicates a more rapid tapering off of driving by women in the upper age groups. This may change, however, as more women who have been familiar with automobiles all of their lives progress into older age groups.

There are no conclusive statistics on the amount of driving by young males as compared to that by young females. However, Illinois, which has shown an active interest in driver license records as a source of information, some time ago added to the license application the question, "Approximately how many miles did you drive a motor vehicle during the past 12 months?" In the analysis of a large sample of the responses, it was found that the males in each age group said they had driven approximately twice as many miles as was reported by female drivers. If this is indicative of driving in the United States as a whole, males do approximately 80 percent of all driving, and females about 20 percent.

The considerable increase in automobile insurance premiums that is invoked in most areas for vehicles driven by male drivers under 25 years of age may have some deterring effect on their applying for licenses, but it is doubtful that this factor could be measured.

The data in table 8 were the primary source of male-female driver license distribution by age groups (figure 1) and were also used in obtaining the estimate of male-female breakdowns for all States that are shown in table 9. The total numbers of drivers' licenses (table 10) were taken from Public Roads table MV–12 for 1963. The distribution of licensed drivers in effect in 1963, by age grouping and by sex, is shown in figure 1; the percentage distribution is shown in table 11.

It may reasonably be expected that the changes in the age distribution of the population, plus the increasing dependency on motor vehicles, will tend to increase the relative proportions of licensed drivers in the older age groups of both sexes. Although somewhat speculative, it also seems probable that the number of licensed female drivers will increase more rapidly than the number of male drivers, until the preponderance of male licensed drivers is substantially less than the 60–40 ratio of 1963.

The total number of driver licenses listed in table 9 are taken from Public Roads table MV–12 (table 10 in this article), which in turn were based on data supplied by the States. In that table, for the States that cannot supply totals of licenses in force, Public Roads estimated the number on the basis of the number issued over a series of years. The male-female segregations in table 9 are those reported by the 19 States that are included in table 8, plus estimates by Public Roads for the remainder. In making the segregations, the most important single factor was the male-female ratio of driver licenses in adjacent States, or in States deemed most similar in characteristics.

Outlook and Forecast

The total population of the United States, the persons of driving age (15 through 74), and the number of licensed drivers are compared in figure 2. The bars for the years through 1960 are from records and estimates, and those for 1965 through 1980 are projections. Worth noting, however, is the fact that the center bar, population in the 15–74 potential driving ages, is close to a statistical certainty. The birth rate after 1965 will not affect the 15–74 age group until 1980; and the effect of even a substantial lengthening of life expectancy could scarcely have an important impact on the totals within this age bracket.

The number of persons licensed to drive, per 1,000 persons in the 15–74 driving age group are shown in figure 3. Figure 3 also shows the number of motor vehicles registered per 1,000 persons in the 15–74 group. The population data and forecasts used in making these computations are basically those of the Bureau of the Census, interpolated by the Bureau of Public Roads where necessary for purposes of this report.

950, in the United States as a whole, there were 593 drivers' licenses per 1,000 people in the 15–74 age group. By 1960, the number had increased to 741, and in 1963, to 764. In the freehand extension of this curve in figure 3, the authors were mindful of the fact that the States have been tending to increase the minimum ages at which drivers licenses may be obtained. Consideration was also given to the fact that the reexamination of drivers reaching specified ages may be expected to have a restraining effect on the number of drivers who might otherwise continue to renew driver licenses despite sight or other physical deficiency. Eleven States now require reexamination of applicants for driver license renewal when the applicant has reached a specified age, and 15 States require vision tests of applicants for renewal. It seems probable that both the mandatory reexamination of older drivers and the retesting of vision at the time of license renewals will be adopted by additional States.

The freehand extrapolation of the drivers license curve indicates about 850 licenses per 1,000 persons in the 15–74 age group by 1980, with the number of licenses per 1,000 persons still increasing, although at a reduced rate.

In 1950, 103 million people were in the 15–74 age group; and there were 62 million drivers' licenses, or approximately 60 percent of the number of people in the group. In the following 13 years, the number of people in the driving age group increased 20 million, and the number of driver licenses increased 32 million. Thus, the number of licenses increased from 60 to 76 percent of the number of people in the 15–74 age group in the short 13-year span.

Table 7.—Grouping of driver license renewal fees, and the average cost[1] per year of drivers' licenses, 1964

Fee group	Number of States in each fee group for license renewal	Number of States having annual driver license cost within the fee groups shown
50 cents or less	1	6
$0.51–$1.00	3	18
$1.01–$1.50	1	8
$1.51–$2.00	9	11
$2.01–$2.50	7	5
$2.51–$3.00	13	2
$3.01–$3.50	1	
$3.51–$4.00	8	1
$4.01–$4.50	1	
$5.00	4	
$6.00	1	
$8.00	1	
TOTAL	50	51

[1] The renewal fees, divided by the number of years for which the license is issued.

The fact that the years 1960–61–62 constitute a flat spot of the drivers license curve of figure 3 is puzzling at first glance. But the greatly increased birth rate that started during World War II is the explanation. It resulted in a wave of teenagers entering the bottom of the 15–74 age group. They were not eligible for driver licenses in some States, and not necessarily licensed at the earliest possible time in the States where they were eligible. The 1963 increase of more than 3 million licenses is at least a preliminary indication that the ratio of drivers' licenses to the number of persons in the 15–74 age group has resumed its upward climb.

If the number of driver licenses do follow the forecast curve in figure 3, there will be more than 110 million by 1970, 125 million by 1975, and 139 million by 1980. Motor-vehicle registrations, 82.7 million in 1963, are projected to reach 110 million by 1975, and exceed 121 million by 1980.

Comparisons among States

Table 12 lists 1963 data about each State, which should be particularly useful. The first and second columns show total population, and population in the 15–74 driving age group. The motor-vehicle registrations listed in the third column include buses and trucks, as well as automobiles. About 70 percent of all trucks are light pickups that often serve as automobiles for personal transportation, as well as for the transportation of goods, and an arbitrary statistical assignment of a portion of the vehicles to personal transportation (versus transportation of goods) would imply a degree of precision that the basic data do not justify. The number of drivers licenses in the fourth column of table 12 are, as mentioned in the discussion of table 8 of this study, taken from Public Roads table MV–12 (table 10), which in turn were derived from the reports of the States.

The number of driver licenses per 1,000 persons in the total population, in the fifth

Table 8.—Drivers' licenses, by sex and age groups, for selected States, 1963

Age	Male	Female	Total	Percent male	Male	Female	Total	Percent male	Male	Female	Total	Percent male	Male	Female	Total	Percent male	Male	Female	Total	Percent male
	Alaska				Colorado				Connecticut				Delaware				Illinois			
Under 16																	6,211	5,369	11,580	53.6
16	785	334	1,119	70.2	45,506	33,519	79,025	57.6	13,167	9,776	22,943	57.4	1,813	1,424	3,237	56.0	61,614	48,772	110,386	55.8
17	960	470	1,430	67.1	27,422	16,758	44,180	62.1	18,600	10,867	29,467	63.1	2,561	1,582	4,143	61.8	70,688	54,271	124,959	56.6
18	1,100	635	1,735	63.4	21,941	13,991	35,932	61.1	14,938	11,775	26,713	55.9	2,056	1,714	3,770	54.5	59,192	43,205	102,397	57.8
19	1,211	831	2,042	59.3	24,812	16,456	41,268	60.1	16,995	14,382	31,377	54.2	2,340	2,094	4,434	52.8	60,151	44,230	104,381	57.6
20–24	7,843	5,300	13,143	59.7	111,295	80,474	191,769	58.0	91,600	73,800	165,400	55.4	18,346	15,328	33,674	54.5	335,669	252,472	588,141	57.1
25–29	8,349	7,040	15,389	54.3	69,476	48,951	118,427	58.7	87,200	68,900	156,100	55.9	18,856	15,598	34,454	54.7	314,727	229,386	544,113	57.8
30–34	8,236	6,923	15,159	54.3	64,260	45,842	110,102	58.4	88,800	71,200	160,000	55.5	18,591	15,330	33,921	54.8	307,746	226,277	534,023	57.6
35–39	8,111	6,339	14,450	56.1	63,964	48,007	111,971	57.1	94,400	77,100	171,500	55.0	17,769	14,777	32,546	54.6	326,703	242,231	568,934	57.4
40–44	7,280	5,523	12,803	56.9	63,379	44,067	107,446	59.0	96,000	75,900	171,900	55.8	16,106	13,536	29,642	54.3	318,341	233,187	551,528	57.7
45–49	6,128	3,921	10,049	61.0	44,016	27,163	71,179	61.8	89,000	65,300	154,300	57.7	13,118	9,627	22,745	57.7	297,973	208,872	506,845	58.8
50–54	4,617	2,573	7,190	64.2	39,096	23,866	62,962	62.1	72,700	49,000	121,700	59.7	10,468	7,097	17,565	59.6	267,918	174,325	442,243	60.6
55–59	3,324	1,585	4,909	67.7	31,333	19,664	50,997	61.4	60,100	36,600	96,700	62.2	8,265	4,710	12,975	63.7	233,360	135,298	368,658	63.3
60–64	1,975	655	2,630	75.1	26,466	15,475	41,941	63.1	45,000	24,000	69,000	65.2	5,993	2,860	8,853	67.7	184,236	92,260	276,496	66.6
65–69	932	222	1,154	80.8	26,172	12,383	38,555	67.9	35,400	16,000	51,400	68.9	6,861	3,255	10,116	67.8	142,255	58,685	200,940	70.8
70 and more	562	105	667	84.3	38,738	11,675	50,413	76.8	40,000	14,100	54,100	73.9	9,887	3,439	13,326	74.2	151,576	41,432	193,008	78.5
TOTAL	61,413	42,456	103,869	59.1	697,876	458,291	1,156,167	60.1	863,900	618,700	1,482,600	58.3	153,030	112,371	265,401	57.7	3,138,360	2,090,272	5,228,632	60.0
	Iowa				Kansas				Kentucky				Minnesota				Montana			
Under 16	505	126	631	80.0	9,886	6,687	16,573	59.7					1,698	709	2,407	70.5	19	8	27	70.4
16	17,811	11,831	29,642	60.1	16,981	13,224	30,205	56.2	18,355	10,484	28,839	63.6	19,675	13,034	32,709	60.2	4,312	2,626	6,938	62.2
17	22,603	16,522	39,125	57.8	17,439	14,108	31,547	55.3	25,930	11,654	37,584	69.0	22,353	16,237	38,590	57.9	4,526	3,010	7,536	60.1
18	18,079	13,768	31,847	56.8	15,872	12,410	28,282	56.1	20,824	12,628	33,452	62.3	23,751	16,407	40,158	59.1	4,463	2,964	7,427	60.1
19	20,456	16,218	36,674	55.8	17,204	13,174	30,378	56.6	23,692	15,423	39,115	60.6	21,168	16,886	38,054	55.6	4,391	3,283	7,674	57.2
20–24	82,895	69,752	152,647	54.3	90,798	64,659	155,457	58.4	107,125	71,900	179,025	59.8	143,542	114,814	258,356	55.6	25,391	22,318	47,709	53.2
25–29	77,119	64,443	141,562	54.5	84,611	57,448	142,059	59.6	97,258	63,441	160,699	60.5	119,491	88,933	208,424	57.3	23,108	20,885	43,993	52.5
30–34	85,300	73,424	158,724	53.7	83,461	58,470	141,931	58.8	89,082	66,261	155,343	57.3	104,610	82,619	187,229	55.9	21,417	18,696	40,113	53.4
35–39	86,407	72,120	158,527	54.5	88,244	61,814	150,058	58.8	90,210	64,851	155,061	58.2	95,982	80,990	176,972	54.2	25,716	21,064	46,780	55.0
40–44	83,511	69,019	152,530	54.8	84,018	60,431	144,449	58.2	88,941	57,943	146,884	60.6	100,451	75,468	175,919	57.1	21,800	17,220	39,020	55.9
45–49	77,783	61,162	138,945	56.0	73,138	53,668	126,806	57.7	77,524	47,369	124,893	62.1	99,356	66,667	166,023	59.8	20,268	16,948	37,216	54.5
50–54	70,568	50,975	121,543	58.1	67,724	48,775	116,499	58.1	70,477	39,193	109,670	64.3	86,041	60,580	146,621	58.7	17,287	13,480	30,767	56.2
55–59	59,790	40,411	100,201	59.7	58,909	42,136	101,045	58.3	59,764	25,236	85,000	70.3	72,445	47,098	119,543	60.6	15,957	10,220	26,177	61.0
60–64	52,783	30,810	83,593	63.1	47,986	32,484	80,470	59.6	47,219	17,481	64,700	73.0	63,351	37,505	100,856	62.8	12,962	7,914	20,876	62.1
65–69	43,548	19,007	62,555	69.6	39,496	23,803	63,299	62.4	30,024	9,587	39,611	76.0	53,220	26,257	79,477	67.0	10,177	5,070	15,247	66.8
70 and more	53,351	12,883	66,234	80.5	58,715	23,204	81,919	71.7	34,252	7,331	41,583	82.4	78,937	25,798	104,735	75.4	19,939	3,911	23,850	83.6
TOTAL	852,509	622,471	1,474,980	57.8	854,482	586,495	1,440,977	59.3	880,677	520,782	1,401,459	62.8	1,106,071	770,002	1,876,073	59.0	231,733	169,617	401,350	57.7
	New York				North Dakota				Ohio				Oklahoma				Oregon			
Under 16					7,904	3,068	10,972	72.0												
16	17,146	8,981	26,127	65.6	5,082	3,196	8,278	61.4	42,210	21,419	63,629	66.3	11,323	7,677	19,000	59.6	6,200	3,000	9,200	67.4
17	46,390	27,214	73,604	63.0	6,051	4,158	10,209	59.3	75,969	23,770	99,739	76.2	17,322	11,678	29,000	82.9	11,800	7,900	19,700	59.9
18	64,616	38,678	103,294	62.6	5,973	4,691	10,664	56.0	58,581	36,589	95,170	61.6	20,671	13,329	34,000	60.8	10,700	8,200	18,900	56.6
19	77,899	47,393	125,292	62.3	5,832	5,158	10,990	53.1	73,776	50,635	124,411	59.3	22,128	13,872	36,000	61.5	10,900	7,500	18,400	59.2
20–24	450,098	297,234	747,332	60.2	23,928	19,344	43,272	55.3	338,330	235,723	574,053	58.9	99,101	70,899	170,000	58.3	57,000	41,500	98,500	57.9
25–29	453,844	299,086	752,930	60.3	19,990	16,037	36,027	55.5	326,102	240,030	566,132	57.6	89,233	65,767	155,000	57.6	55,800	40,200	96,000	58.1
30–34	488,979	326,571	815,550	60.0	19,216	15,781	34,997	54.9	315,212	224,516	539,728	58.4	77,333	59,667	137,000	56.4	55,200	47,200	102,400	53.9
35–39	528,018	365,493	893,511	59.1	19,317	15,034	34,351	56.2	343,176	256,560	599,736	57.2	70,626	55,374	126,000	56.1	62,600	49,100	111,700	56.0
40–44	516,986	352,618	869,604	59.5	18,578	14,382	32,960	56.4	319,327	229,387	548,714	58.2	70,050	53,950	124,000	56.5	61,200	53,200	114,400	53.5
45–49	478,074	311,073	789,147	60.6	19,192	13,243	32,435	59.2	291,870	207,294	499,164	58.5	64,294	48,706	113,000	56.9	58,000	44,200	102,200	56.8
50–54	440,715	257,991	698,706	63.1	16,844	12,271	29,115	57.9	244,481	159,183	403,664	60.6	60,230	44,770	105,000	57.4	53,500	35,800	89,300	59.9
55–59	395,231	206,744	601,975	65.7	15,197	9,231	24,428	62.2	220,820	132,657	353,477	62.5	54,930	37,070	92,000	59.7	45,600	29,300	74,900	60.9
60–64	319,315	141,707	461,022	69.3	12,314	6,305	18,619	66.1	158,401	88,389	246,790	64.2	44,537	27,463	72,000	61.9	38,400	20,000	58,400	65.8
65–69	245,046	98,943	343,989	71.2	10,242	4,032	14,274	71.8	131,401	57,953	189,354	69.4	31,640	16,360	48,000	65.9	30,600	12,300	42,900	71.3
70 and more	272,257	90,051	362,308	75.1	14,362	2,930	17,292	83.1	149,735	46,465	196,200	76.3	40,007	14,993	55,000	72.7	40,600	11,500	52,100	77.9
TOTAL	4,794,614	2,869,777	7,664,391	62.6	220,022	148,861	368,883	59.6	3,089,391	2,010,570	5,099,961	60.6	773,425	541,575	1,315,000	58.8	598,100	410,900	1,009,000	59.3
	South Carolina				South Dakota				Texas				Wisconsin				Total, 19 States			
Under 16	5,204	2,705	7,909	65.8	8,195	3,409	11,604	70.6	50,289	36,726	87,015	57.8					153,447	95,002	248,449	61.8
16	12,184	6,518	18,702	65.2	5,407	3,292	8,699	62.2	45,021	30,522	75,543	59.6	17,408	10,042	27,450	63.4	351,299	233,819	585,118	60.0
17	16,594	7,502	24,096	68.9	6,437	4,282	10,719	60.1	66,700	44,965	111,665	59.7	25,864	16,684	42,548	60.8	466,939	287,134	754,073	61.9
18	14,670	6,956	21,626	67.8	5,953	3,954	9,907	60.1	69,156	44,595	113,751	60.8	12,687	10,242	22,929	55.3	430,364	286,727	717,091	60.0
19	15,217	7,941	23,158	65.7	5,813	4,347	10,160	57.2	72,654	45,546	118,200	61.6	22,584	15,105	37,689	59.9	480,517	326,633	807,150	59.5
20–24	80,453	49,469	129,922	61.9	24,050	21,140	45,190	53.2	329,971	236,070	566,041	58.3	116,398	88,460	204,858	56.8	2,533,833	1,830,656	4,364,489	58.1
25–29	76,809	52,781	129,590	59.3	21,906	19,798	41,704	52.5	321,401	236,881	558,282	57.6	105,458	85,765	191,223	55.2	2,370,738	1,721,370	4,092,108	57.9
30–34	73,218	53,462	126,680	57.8	21,405	18,686	40,091	53.4	320,248	247,090	567,338	56.5	92,324	79,676	172,000	53.7	2,334,638	1,737,691	4,072,329	57.3
35–39	75,246	54,469	129,715	58.0	20,666	16,928	37,594	55.0	320,272	251,111	571,383	56.1	112,684	91,096	203,780	55.3	2,450,111	1,844,458	4,294,569	57.1
40–44	74,312	50,212	124,524	59.7	19,384	15,312	34,696	55.9	292,246	225,075	517,321	56.5	117,931	84,885	202,816	58.2	2,369,841	1,731,315	4,101,156	57.8
45–49	64,506	39,515	104,021	62.0	18,157	15,182	33,339	54.5	262,470	198,837	461,307	56.9	116,646	74,985	191,631	60.9	2,171,513	1,513,732	3,685,245	58.9
50–54	55,365	31,897	87,262	63.5	16,540	12,897	29,437	56.2	236,692	175,939	412,631	57.4	101,014	68,139	169,153	59.7	1,932,277	1,268,751	3,201,028	60.4
55–59	41,977	22,639	64,616	65.0	14,994	9,603	24,597	61.0	197,708	133,423	331,131	59.7	93,054	59,655	152,709	60.9	1,682,758	1,003,280	2,686,038	62.7
60–64	30,171	13,938	44,109	68.4	12,232	7,468	19,700	62.1	151,076	93,156	244,232	61.9	81,373	47,505	128,878	63.1	1,335,790	707,375	2,043,165	65.4
65–69	21,569	9,235	30,804	70.0	10,684	5,322	16,006	66.8	117,487	60,747	178,234	65.9	56,440	23,480	79,920	70.6	1,043,194	462,641	1,505,835	69.3
70 and more	24,838	7,798	32,636	76.0	15,481	3,037	18,518	83.6	135,811	50,898	186,709	72.7	83,714	23,070	106,784	78.4	1,262,762	394,620	1,657,382	76.2
TOTAL	682,333	417,037	1,099,370	62.1	227,304	164,657	391,961	58.0	2,989,202	2,111,581	5,100,783	58.6	1,155,579	778,789	1,934,368	59.7	23,370,021	15,445,204	38,815,225	60.2

column, reflect the combined effects of the age composition of the populations of the States, and the extent of driver license saturation in each. Mississippi, in 1963, had only 353 licensed drivers per 1,000 persons in the total population. The next lowest was Louisiana, having 397. At the other extreme, Wyoming had 672 licensed drivers per 1,000 persons, and Kansas had 652.

Perhaps the most significant information in table 12 is the number of licensed drivers per 1,000 persons in the 15–74 age group. The Kansas total of 992 driver licenses per 1,000 persons in the driving age group, is not much higher than the 971 in Wyoming and the 968 in Colorado, and many other States have more than 900. The lowest number of driver licenses per 1,000 persons in the 15–74 group was 629 in Mississippi; next were the 635 in the District of Columbia and 637 in Alaska. The number of driver licenses per 1,000 persons of driving age in the other States were broadly distributed between these highs and lows, without any obvious pattern.

The ratios of driver licenses to persons in the 15–74 groups raise some interesting questions. To begin with, some of the ratios would seem improbable, if we were to assume that all licenses were held by persons residing in the State and included in the Census Bureau population counts. Granting that the ability and desire to drive a motor vehicle is almost universal, it is difficult to conceive of circumstances that would result in a number of driver licenses in any State that is equal to 992 out of each 1,000 persons in the 15–74 age group. In Kansas, for which the statistics yield this rate, 16 is the minimum age at which a license can be obtained. The need to examine the statistics is obvious.

Discussion of Limitations and Summary

There were two reasons for selecting the 15–74 age group for many of the essential statistical comparisons made in this study: (1) The State-by-State population data were readily available from the Bureau of the Census, and (2) the age grouping is one in which we should expect to count all but a very few active motor vehicle operators. The 15-year terminal is reasonably close to the minimum driving ages permitted by the States, and there is undoubtedly a statistical balancing of persons who do not obtain driver licenses at age 15 by persons more than 74 years old who retain their licenses.

Undoubtedly, many persons hold valid driver licenses from more than one State. Even if this were not legal, there is no present procedure for eliminating the duplication caused by persons who move from one State to another, and who obtain another license, provided, of course, that the person has a good driving record. The requirements of the States for the obtaining of a driver's license by a new resident, coupled with the absence of any effective general ban on a driver continuing to renew licenses previously obtained in other States, tend to increase the totals of drivers' licenses. Although it seems probable that holding of multiple driver licenses is fairly common, examination of the ratios of licenses to motor-vehicle registrations, in the last column of table 12, makes it seem doubtful that the total resultant duplication could be very large. Indeed, another question is raised. Are there any substantial number of vehicles operated by unlicensed drivers, including persons who have allowed their licenses to expire? Possession of a driver's license is not a prerequisite to ownership or licensing of a vehicle in any State. No effort was made in this study to analyze the possible statistical effects of the holding of multiple licenses, nor to determine the extent of enforcement of the requirement of a license. However, it is not suggested that a study of these factors would be likely to yield conclusive findings.

The extent or diligence of enforcement of the license requirements for operators of motor vehicles, coupled with the severity of penalties for noncompliance, undoubtedly have some effect on the number of driver licenses issued. Probably of much greater importance, however, is the ease or difficulty of obtaining and retaining a license. Where the requirements are rigid, and strictly enforced, the process could be expected to reduce the number of licenses issued by screening out potentially incompetent or otherwise hazardous drivers. A ratio of licenses to persons in the 15-74 age group that is lower in one State than in another of similar characteristics might easily result from differences in the requirements of the States for obtaining licenses. This study does attempt to detect or measure the extent of any such effect, but the information in tables 1, 4, 5, and 6 may be useful for those who wish to do so The tables, however, are of no assistance in determining the extent to which differenees in the interpretations and strictness of enforcement among the States may affect either the number of licenses issued or the statistical strueture of the driver-licensed population.

The fact that the mere grouping of number, sex, and age of licensed drivers does not furnish any information on driving exposure by the groups is so obvious that the authors are reluctant to state it. We do so only to suggest that it would be highly desirable for others to correlate the information in this report with available data on the miles driven by persons in corresponding age and sex groups. Such a study should prove to be extremely useful for many purposes.

The data and estimates presented in this report are admittedly less than might be desired, but they do produce some relationships between the number of driver licenses,

Table 9.—Estimated total drivers' licenses in force, by State and sex, 1963

State	Male drivers	Female drivers	Total drivers	Percentage male drivers of total
	Thous.	*Thous.*	*Thous.*	*Percent*
Alabama	891	587	1,478	60.3
Alaska	62	42	104	59.1
Arizona	534	361	895	59.7
Arkansas	531	350	881	60.3
California	5,367	3,686	9,053	59.3
Colorado	698	458	1,156	60.4
Connecticut	864	619	1,483	58.3
Delaware	153	112	265	57.7
Florida	1,906	1,167	3,073	62.0
Georgia	1,268	835	2,103	60.3
Hawaii	261	180	441	59.1
Idaho	240	162	402	59.8
Illinois	3,139	2,090	5,229	60.0
Indiana	1,507	993	2,500	60.3
Iowa	853	622	1,475	57.8
Kansas	855	586	1,441	59.3
Kentucky	880	521	1,401	62.8
Louisiana	819	539	1,358	60.3
Maine	286	177	463	61.9
Maryland	1,001	613	1,614	62.0
Massachusetts	1,515	934	2,449	61.9
Michigan	2,360	1,617	3,977	59.4
Minnesota	1,106	770	1,876	59.0
Mississippi	487	321	808	60.3
Missouri	1,368	900	2,268	60.3
Montana	232	169	401	57.7
Nebraska	514	363	877	58.6
Nevada	123	83	206	59.8
New Hampshire	224	138	362	61.9
New Jersey	1,960	1,209	3,169	61.9
New Mexico	317	213	530	59.8
New York	4,794	2,870	7,664	62.6
North Carolina	1,417	867	2,284	62.0
North Dakota	220	149	369	59.7
Ohio	3,089	2,011	5,100	60.6
Oklahoma	774	541	1,315	58.8
Oregon	598	411	1,009	59.3
Pennsylvania	3,620	2,232	5,852	61.9
Rhode Island	273	168	441	61.9
South Carolina	682	417	1,099	62.1
South Dakota	227	165	392	58.0
Tennessee	1,071	705	1,776	60.3
Texas	2,989	2,112	5,101	58.6
Utah	328	220	548	59.8
Vermont	120	74	194	61.9
Virginia	1,250	765	2,015	62.0
Washington	897	616	1,513	59.3
West Virginia	539	355	894	60.3
Wisconsin	1,155	779	1,934	59.7
Wyoming	135	91	226	59.8
Dist. of Col	211	136	347	60.5
TOTAL	56,710	37,101	93,811	60.5

Table 10.—Motor-vehicle operators' and chauffeurs' licenses, 1963 [1]

[Compiled for calendar year from reports of State authorities]

State	Learners' permits [2]		Operators' licenses						Chauffeurs' licenses [4]						Estimated total licenses in force during 1963 [5]	Private and commercial motor vehicles registered in 1963	Licensed operators per registered motor vehicle
	Issued during 1963	Amount of fee	Issued during 1963	Issued for, years	Renewal date	Amount of fee [3]			Issued during 1963	Issued for, years	Renewal date	Amount of fee [3]					
						New	Renewal	Duplicate				New	Renewal	Duplicate			
Alabama	79,197	$0.35	720,082	2	Birthday	$4.25	$4.25	$0.25		Not required.					1,478,420	1,435,359	1.03
Alaska	4,900	1.00	29,072	3	Birthday	5.00	5.00	1.00	[6] 175	1	Sept. 2	$2.00			103,869	89,412	1.16
Arizona	24,721	*2.00	271,903	3	Birthday	2.50	2.50	1.00	48,351	2	Birthday	2.50	$2.50	$1.00	895,299	733,505	1.22
Arkansas	N.A.	No fee	846,249	1	Jan. 1	2.00	2.00	2.00	34,277	1	Jan. 1	5.00	5.00	1.00	880,526	819,327	1.07
California	420,108	[7] 3.00	[8] 2,732,190	3 and 5	Birthday	3.00	3.00	1.00	[8] 40,641	3 and 5	Birthday	3.00	3.00	1.00	9,053,189	8,983,975	1.01
Colorado	33,294	[7] 2.25	358,872	3	Birthday	2.25	2.25	1.25	37,825	3	Birthday	5.25	5.25	1.25	1,156,167	1,052,104	1.10
Connecticut			809,250	2	Birth mo	(9)	6.00	1.00	8,500	1	May 1	3.00	3.00	1.00	*1,482,600	1,261,770	1.18
Delaware	23,004	[7] 4.00	48,771	(10)	Birthday	4.00	4.00	1.00	5,097	(10)	Birthday	4.00	4.00	1.00	265,401	219,836	1.21
Florida	137,862	3.00	1,288,741	2	Birth mo	3.00	3.00	.25	169,529	2	Birth mo	5.00	5.00	.25	3,073,245	2,695,829	1.14
Georgia	83,887	1.00	966,333	1 or 5	Birthday	[11] 1.00	1.00	1.00	98,493	1 or 5	Birthday	[11] 2.00	2.00	1.00	2,102,442	1,752,035	1.20
Hawaii	33,885	*1.00	47,132	Indefinite		3.00		.50	3,803	1	Issuance	3.00	1.00	.50	*441,298	265,842	1.66
Idaho	11,295	[12] 3.00	206,300	2	Birthday	4.00	4.00	.75	18,660	1	Birthday	3.00	3.00	.75	402,143	398,161	1.01
Illinois	234,549	[7] 3.00	1,424,121	3	Birthday	3.00	3.00	1.00	307,116	1	Issuance	5.00	3.00	1.00	5,228,632	4,064,791	1.29
Indiana	77,920	*1.00	884,719	2	Birth mo	1.50	1.50	1.50	[13] 234,179	1	Birth mo	1.50	1.50	1.50	2,499,735	2,232,258	1.12
Iowa	51,377	3.00	671,595	2	Birthday	3.00	3.00	.25	153,634	1	Birthday	4.00	4.00	.50	1,474,980	1,414,312	1.04
Kansas	43,272	.50	626,011	2	Birthday	[14] 5.00	2.00	.50	59,727	2	Birthday	[14] 7.00	4.00	.50	1,440,977	1,261,771	1.14
Kentucky	139,975	1.00	650,177	2	Birth mo	2.00	2.00	1.00	18,725	1	Jan. 1	2.00	2.00	2.00	*1,401,459	1,334,723	1.05
Louisiana	N.A.	[7] 2.50	665,678	2	Birthday	2.50	2.50	1.50	145,591	1	Issuance	3.50	3.50	1.50	1,358,046	1,281,479	1.06
Maine	15,991	3.00	463,465	1	Birthday	2.00	2.00	.50		Not required.					463,465	401,687	1.15
Maryland	164,959	* [7] 5.00	751,037	2	Birth mo	*7.00	*2.00	.50	56,343	2	Issuance	*8.00	*3.00	.50	*1,613,746	1,301,408	1.24
Massachusetts			1,387,612	2	Birthday	[14] 8.00	5.00	1.50	[6] 2,693	1	Issuance	[14] 5.50	2.50	1.50	*2,448,517	1,955,851	1.25
Michigan	N.A.	No fee	1,431,761	3	Birthday	4.00	2.50	1.00	242,572	1	Issuance	4.00	2.75	1.00	*3,977,165	3,559,044	1.12
Minnesota	149,351	1.00	453,534	4	Birthday	3.00	3.00	.50	211,882	1	Feb. 1	3.00	[15] 2.00	1.00	*1,876,073	1,707,037	1.10
Mississippi	11,973	.50	525,911	1 or 2	Issuance	* [16] 2.50	* [16] 2.50	1.00	43,545	1 or 2	Issuance	*[16] 4.50	*[16] 4.50	1.00	808,199	807,238	1.00
Missouri	201,861	.25	465,445	3	Issuance	1.00	1.00	.25	192,663	1	Issuance	3.00	3.00	.25	2,268,392	1,880,346	1.21
Montana	5,733	*[7] 4.00	147,664	2	Birthday	4.00	4.00	.50	23,929	2	Birthday	4.00	4.00	.50	401,350	411,090	0.98
Nebraska	47,542	1.00	742,381	2	Sept. 1 odd year.	3.00	3.00	.50		Not required.					877,385	798,589	1.10
Nevada	5,435	[7] 3.00	118,202	5	Birthday	3.00	3.00	1.00	12,483	5	Birthday	5.00	5.00	1.00	205,885	235,162	0.88
N. Hampshire			101,461	2	Birthday	5.00	4.00	1.00	12,007	2	Birthday	7.00	4.00	1.00	361,547	280,030	1.29
New Jersey	381,820	*2.00	2,089,223	1 or 3	Issuance	[17] 3.00	[17] 3.00	2.00	[18] 14,790	Indefinite		No fee			*3,168,858	2,710,969	1.17
New Mexico	15,817	.50	223,699	2	Birth mo	3.25	3.25	1.00	63,971	1	Birth mo	2.75	2.75	1.00	529,791	473,028	1.12
New York	499,783	.50	1,879,550	3	Issuance	[14] 5.00	3.00	3.00	387,283	3	Issuance	[14] 8.00	6.00	3.00	7,664,391	5,455,455	1.40
North Carolina	123,913	No fee	569,982	4	Birthday	2.50	2.50	.50	73,187	2	Birthday	4.00	4.00	.50	2,283,576	1,891,847	1.21
North Dakota	N.A.	[7] 3.00	342,356	2	Birth mo	*[19] 3.00	*3.00	1.00		Not required.					368,883	369,216	1.00
Ohio	391,744	*.50	2,528,605	3	Birthday	*.75	*.75	*.25	219,434	3	Birthday	*1.25	*1.25	*[20] .25	5,099,961	4,424,528	1.15
Oklahoma	15,380	No fee	599,731	2	Birth mo	4.00	4.00	1.00	[13] 81,802	2	Birth mo	8.00	8.00	1.00	1,315,000	1,313,615	1.00
Oregon	61,905	.50	539,592	2	Birthday	(21)	2.75	.25	26,381	2	Birthday	2.00	2.00	.25	1,009,000	994,804	1.01
Pennsylvania	349,489	*[7] 4.00	4,080,369	2	Feb. 1	4.00	4.00	.50		Not required.					5,851,840	4,571,750	1.28
Rhode Island	21,335	No fee	203,289	2	Oct. 1	[14] 13.00	8.00	1.00	12,534	2	Oct. 1	[14] 13.00	8.00	1.00	440,726	369,352	1.19
South Carolina	66,785	No fee	79,571	4	July 1	.50	.50	.50	[22] 6,811	1	Jan. 1	2.00	2.00	No fee	*1,099,370	956,151	1.15
South Dakota	N.A.	2.00	73,009	4	Birthday	2.00	2.00	2.00		Not required.					391,961	381,833	1.03
Tennessee	22,890	[7] 4.00	1,562,678	2	Birthday	4.00	4.00	2.00	95,452	2	Birthday	6.00	6.00	2.00	1,775,960	1,476,876	1.20
Texas	138,177	* [7] 3.00	[13] 2,658,095	2	Issuance	3.00	3.00	.25	150,466	1	Issuance	6.00	6.00	.25	5,100,783	5,009,008	1.02
Utah	24,823	[7] 3.00	124,287	5	Birthday	[17] 3.00	2.00	1.00	5,415	5	Birthday	[17] 3.00	2.00	1.00	547,952	476,642	1.15
Vermont	5,520	1.00	193,807	1	Birthday	[14] 4.50	2.50	.50		Not required.					193,807	160,675	1.21
Virginia	N.A.	No fee	815,492	3	Birth mo	2.00	2.00	.25	88,601	1	Issuance	3.00	3.00	.25	2,014,448	1,619,474	1.24
Washington	184,492	[7] 1.50	801,867	2	Birthday	[14] 6.00	4.00	.50		Not required.					1,513,328	1,498,386	1.01
West Virginia	55,368	4.00	202,377	4	Issuance	5.00	5.00	1.00	43,251	1	Issuance	3.00	3.00	1.00	893,597	636,160	1.40
Wisconsin	148,435	1.50	991,225	2	Birthday	[14] 4.50	2.00	1.00	[6] 68,517	1	Birthday	[14] 5.00	2.00	1.00	1,934,368	1,711,987	1.13
Wyoming	6,523	No fee	54,358	3	Birthday	2.00	2.00	1.00	15,309	1	Issuance	2.00	2.00	1.00	226,438	212,879	1.06
Dist. of Col	46,034	*2.00	108,513	3	Issuance	3.00	3.00	.50		Not required.					347,193	211,954	1.64
TOTAL															93,811,383	81,560,560	1.15

[1] Complete data for all States were not available.

[2] In many States an instruction or learner's permit is provided but is not required except under certain circumstances. Only in the States indicated with an asterisk (*) is such a permit mandatory for applicants not possessing a valid operator's license. An instruction or learner's permit is not provided in the States for which leaders (---) appear. The number of permits issued is not available in the States for which N.A. appears.

[3] In the States marked with an asterisk (*) an additional nominal local service charge is collected at the time the license is issued.

[4] Includes public service and other special licenses that are issued to operators of vehicles for hire.

[5] Estimated by the Bureau of Public Roads from data reported by the States for current and previous years. No allowance was made for deaths, emigrations, or revocations. Chauffeurs' licenses have not been added to operators' licenses in the States that require an operator's license in addition to the chauffeur's license. Such States are indicated with an asterisk (*).

[6] Special licenses to operate school buses. In Wisconsin 5,261 are included at $2.50 each for a 2-year term.

[7] Permit fee is credited to operator license fee; in Pennsylvania $2 and in Washington $0.50 of permit fee is credited to operator license fee.

[8] Since September 15, 1961, drivers' licenses have not been designated as *Operator* or *Chauffeur* licenses by the State. The applicant is required to take an examination appropriate to the type of motor vehicle he will operate, class 1, 2, or 3. The class 3 license corresponds to the former *Operator* license.

[9] $5 examination fee, plus 25 cents per month from date of issuance to last day of month of next birthday, plus $3 or $6, depending upon year of birth.

[10] Licenses are issued for a 2-year period, but drivers meeting certain requirements and having a motor-vehicle operation record that shows no previous arrest or conviction can obtain licenses for an indefinite period—$10.

The number of licenses issued were:

	2-years	*Indefinite*	*Total*
Operators	70,288	1,487	71,775
Chauffeurs	4,912	185	5,097

[11] Operator's fee is $5 for 5 years and chauffeur's fee is $10 for 5 years. Free licenses to veterans.

[12] Every applicant for an instruction permit or operator's license who is required to take or who elects to take a driver training course in a public school is required to pay an addition fee of $3.

[13] Special commercial licenses are included as follows: 20,486 public passenger in Indiana, 43,821 commercial chauffeur in Oklahoma at $10 each, and 832,621 commercial operator in Texas at $4.50 each, 1-year term.

[14] The difference between new and renewal license fees is the charge for examination when one is required, except in Wisconsin, where the examination fee is $2.

[15] Chauffeurs' licenses renewed during February are $2, thereafter $2.50. Schoolbus operators' licenses (which are also valid chauffeurs' licenses) renewed during May are $2, thereafter $2.50.

[16] Option of obtaining 1- or 2-year permits at $2.50 per year for operator's license and $4.50 a year for chauffeur's license.

[17] 3-year license, $8 in New Jersey; original license issued for a 3-year term in Utah.

[18] Special bus drivers licenses. Issued for an indefinite period, but evidence of physical fitness, good character, and experience is required every 12 months.

[19] License fee for those under 18 is $1.50.

[20] An additional $0.50 is charged if the chauffeur's badge also needs to be replaced.

[21] Original license fees vary, $2.25, $2.75, or $3.25, depending upon length of time from date of application to date of first renewal.

[22] A permit to operate "For hire" vehicles is required.

Table 11—Distribution of licensed drivers, by sex and by percentage in each age group 1963

Age	Male drivers	Female drivers	All drivers
Years	*Percent*	*Percent*	*Percent*
Under 16	0.3	0.3	0.3
16	1.6	1.6	1.6
17	2.2	1.8	2.0
18	1.8	1.8	1.8
19	2.0	2.1	2.0
(Under 20)	(7.9)	(7.6)	(7.7)
20–24	10.8	11.8	11.2
25–29	10.1	11.1	10.5
30–34	9.9	11.4	10.5
35–39	10.5	12.1	11.1
40–44	10.2	11.5	10.7
45–49	9.4	9.9	9.6
50–54	8.3	8.2	8.3
55–59	7.2	6.5	6.9
60–64	5.8	4.5	5.2
65–69	4.5	2.9	3.9
70 and over	5.4	2.5	4.4
TOTAL	100.0	100.0	100.0

population, and motor-vehicle registrations. Some data were obtained from records not well adapted to the extraction of statistics, or lacking the review and refinement that would have been desirable to assure complete accuracy and consistency. Many of the State offices that administer drivers' licenses suffer from a numbing lack of manpower and equipment to do such a job.

Yet even with these shortcomings, the report constitutes a sort of breakthrough. It attempts a summary of more information on the number, age, and sex of licensed drivers than has been found elsewhere; and it should be an encouragement to those who have use for such information. It is hoped that this, in turn, will bring about further steps toward providing the means to extract the data from driver license records, so that its tremendous potential value to government, business, and the general public may be fully realized.

Table 12.—Ratio of drivers' licenses in force to population and motor vehicles, 1963

State	Total population	Persons of driving age	Vehicle registration	Drivers' licenses			
				Number	Per 1,000 total population	Per 1,000 driving population	Per vehicle
	Thous.	*Thous.*	*Thous.*	*Thous.*	*Number*	*Number*	*Number*
Alabama	3,347	2,089	1,435	1,478	442	708	1.03
Alaska	248	163	90	104	419	637	1.16
Arizona	1,559	938	734	895	574	955	1.22
Arkansas	1,858	1,110	819	881	474	793	1.07
California	17,590	11,998	8,984	9,053	515	755	1.01
Colorado	1,961	1,195	1,052	1,156	590	968	1.10
Connecticut	2,666	1,788	1,262	1,483	556	829	1.18
Delaware	476	315	220	265	558	843	1.21
Florida	5,652	3,746	2,696	3,073	544	820	1.14
Georgia	4,140	2,607	1,752	2,103	508	807	1.20
Hawaii	694	467	266	441	636	945	1.66
Idaho	713	426	398	402	564	944	1.01
Illinois	10,182	6,966	4,065	5,229	514	751	1.29
Indiana	4,694	3,135	2,232	2,500	533	797	1.12
Iowa	2,780	1,782	1,414	1,475	531	828	1.04
Kansas	2,225	1,452	1,262	1,441	652	992	1.14
Kentucky	3,095	1,958	1,335	1,401	453	716	1.05
Louisiana	3,418	2,095	1,281	1,358	397	648	1.06
Maine	982	634	402	463	472	731	1.15
Maryland	3,289	2,158	1,301	1,614	491	748	1.24
Massachusetts	5,218	3,509	1,956	2,449	469	698	1.25
Michigan	8,116	5,245	3,559	3,977	490	758	1.12
Minnesota	3,500	2,215	1,707	1,876	536	847	1.10
Mississippi	2,290	1,284	807	808	353	629	1.00
Missouri	4,328	2,910	1,880	2,268	524	780	1.21
Montana	707	436	411	401	568	921	0.98
Nebraska	1,460	919	799	877	601	955	1.10
Nevada	368	218	235	206	560	944	0.88
New Hampshire	627	410	280	362	577	882	1.29
New Jersey	6,470	4,351	2,711	3,169	490	728	1.17
New Mexico	1,018	626	473	530	520	846	1.12
New York	17,708	11,863	5,455	7,664	433	646	1.40
North Carolina	4,760	2,998	1,892	2,284	480	762	1.21
North Dakota	634	390	369	369	582	946	1.00
Ohio	10,173	6,604	4,425	5,100	501	772	1.15
Oklahoma	2,487	1,545	1,314	1,315	529	851	1.00
Oregon	1,826	1,187	995	1,009	553	850	1.01
Pennsylvania	11,424	7,720	4,572	5,852	512	758	1.28
Rhode Island	885	590	369	441	498	747	1.19
South Carolina	2,483	1,516	956	1,099	443	725	1.15
South Dakota	737	424	382	392	532	924	1.03
Tennessee	3,694	2,358	1,477	1,776	480	753	1.20
Texas	10,323	6,489	5,009	5,101	494	786	1.02
Utah	983	574	477	548	557	955	1.15
Vermont	390	251	161	194	497	772	1.21
Virginia	4,331	2,705	1,619	2,015	465	745	1.24
Washington	3,050	1,931	1,498	1,513	496	784	1.01
West Virginia	1,778	1,158	636	894	503	772	1.40
Wisconsin	4,060	2,606	1,712	1,934	476	742	1.13
Wyoming	337	233	213	226	672	971	1.06
District of Columbia	797	547	212	347	436	635	1.64
TOTAL	188,531	122,834	81,561	93,811	498	764	1.15

New Publications

Three new publications have been issued recently by the Bureau of Public Roads, U.S. Department of Commerce. All are for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402. The price for each is as indicated and should be included with your order: *A Quarter Century of Financing Municipal Highways, 1937–61, $1.00; Traffic Assignment Manual,* $1.50; *Accidents on Main Rural Highways Related To Speed, Driver, and Vehicle, 35 cents.* A brief discussion of each of the new publications is presented in the following paragraphs.

A Quarter Century of Financing Municipal Highways, 1937–61

The first comprehensive survey of highway receipts, disbursements, and debt transactions of municipal governments during the past 25 years is presented in *A Quarter Century of Financing Municipal Highways, 1937–61.* Information is presented primarily on the highway revenues and expenditures of municipal governments. Data on the finances of State and county governments as they pertain to municipal areas have been included for comparative purposes. To broaden the area of coverage, estimates have been included for the 1921–36 period in the summary data.

This publication consists of two sections: The first contains summary tables and a discussion of municipal highway finance; the second (appendix) contains the detailed State-by-State tabulations upon which the summary tables were based. The tabulations of the highway finances of municipal governments that have been published in the Bureau of Public Roads annual *Highway Statistics* series since 1949, are superseded by the data in this publication. *A Quarter Century of Financing Municipal Highways, 1937–61* has been issued as a companion publication to the *Financing of Highways by Counties and Local Rural Governments, 1942–51.*

Traffic Assignment Manual

Issued as a companion publication to *Calibrating and Testing a Gravity Model for Any Size Urban Area,* which is also for sale by the Superintendent of Documents, the *Traffic Assignment Manual* documents the complete process of traffic assignment as currently defined and primarily is for use with the IBM 7090/94 computer. A brief history and general concepts of traffic assignment are presented, and the step-by-step procedures for each phase of the traffic assignment process are given. Also included is a discussion of the theory involved in the different phases of traffic assignment, a detailed glossary of terms, and a selected bibliography. The operation of the BELL and BELMN control monitors are described in detail, and the complete battery of Public Roads traffic assignment programs are included. This manual and the gravity model manual are expected to provide transportation planners with the necessary tools for comprehensive transportation planning studies.

Accidents on Main Rural Highways Related to Speed, Driver, and Vehicle

Significant information on the relation of speed and characteristics of vehicles and drivers involved in accidents on main rural highways is presented in *Accidents on Main Rural Highways Related to Speed, Driver, and Vehicle.* It is believed that the material presented in this publication is the first based on a nationwide study from which it has been possible to develop an understanding of these relationships. The study findings apply to 2- and 4-lane main rural highways of the nonfreeway type.

One of the important findings reported in this publication is that the greater the differential in speed of a driver and his vehicle from the average speed of all traffic, the greater the chance of that driver being involved in an accident. For example, a driver traveling at 40 or 80 miles per hour in relation to an average speed of 60 miles per hour for all traffic has a substantially greater chance of being involved in an accident than a driver traveling at the average speed.

Errata

In vol. 32, No. 9, August 1964, PUBLIC ROADS, *Effect of Moisture on Bituminous Pavements in Rocky Mountain Areas,* change last three sentences in first column of page 51 to read: Seal coats cost about 11 cents per square yard or, on a 3-inch thick mat, about an additional 70 cents per ton of mixture. This is more expensive than either the chemical additive, at 11 cents per ton of mixture, or hydrated lime, at 45 cents per ton of mixture. Furthermore, the seal coat may not always solve the problem, as there are so many uncertainties involved in seal coat construction as to make the results somewhat of a gamble.

U.S. GOVERNMENT PRINTING OFFICE: 1964

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Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:

First Progress Report, House Document No. 106 (1957). 35 cents.

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

Final Report, Part VI: Economic and Social Effects of Highway Improvement, House Document No. 72 (1961). 25 cents.

The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways, 1937–61. $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

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The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Aerial Surveys in Highway Engineering (1960). 40 cents.

Traffic Assignment Manual. $1.50.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Transition Curves for Highways (1940). $1.75.

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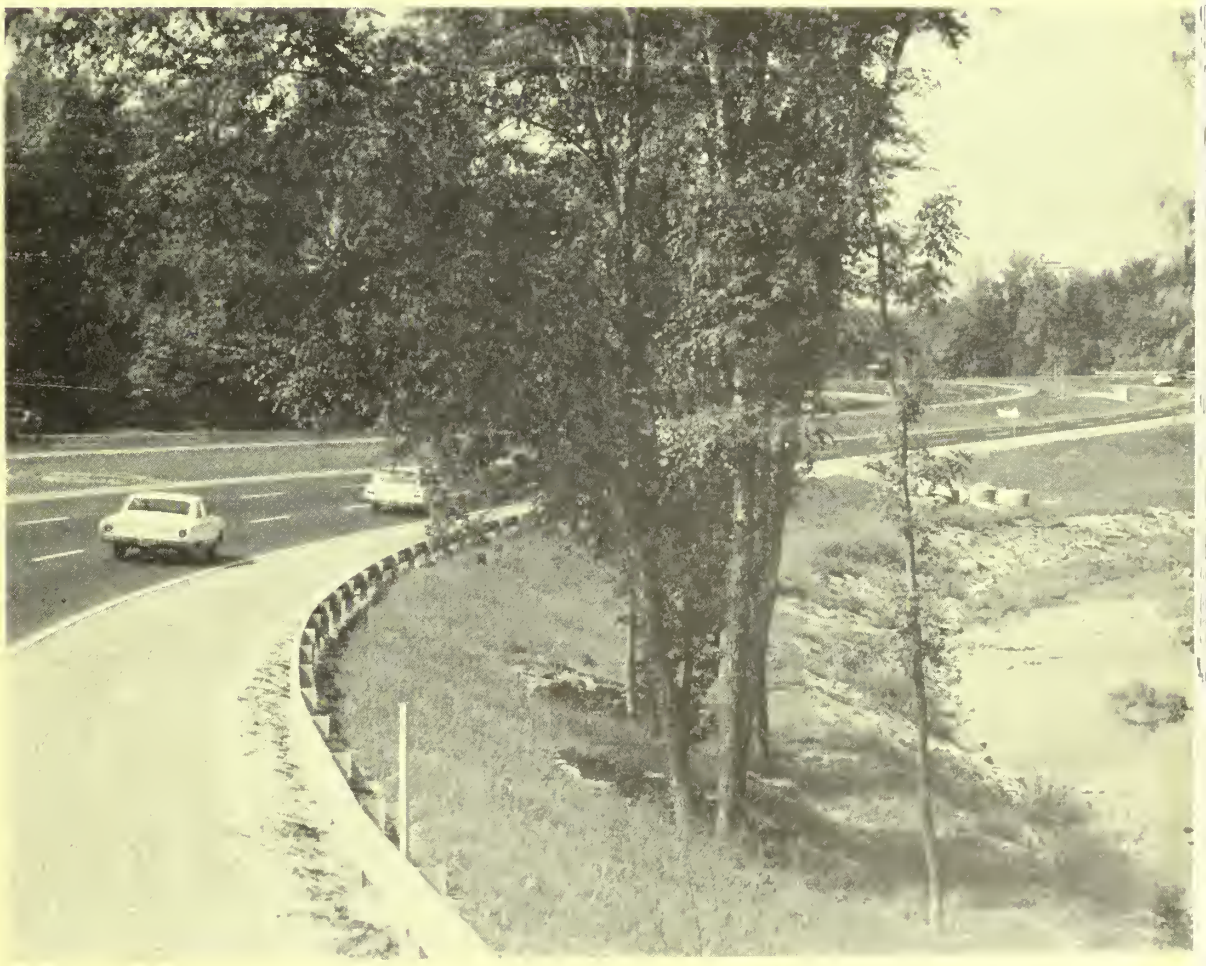

Scene between interchanges 20 and 19, Connecticut and Wisconsin Avenues, on the Maryland portion of the Capital Beltway, Interstate Highway 495

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 33, No. 5 *December 1964*

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Jessie W. Dean, *Assistant Editor*

IN THIS ISSUE

Timesharing Between Two Driving Tasks—Simulated Steering and Recognition of Road Signs, *by B. W. Stephens and R. M. Michaels* . 81

Evaluation of Laboratory Vane Shear Test, *by R. D. Goughnour and J. R. Sallberg* 89

Evaluation of Results of Laboratory Vane Shear Tests With Pore-Pressure Measurements, *by N. E. Wilson* 91

New Publications 96

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Content of this publication may be reprinted. Mention of source is requested.

Timesharing Between Two Driving Tasks

Simulated Steering and Recognition of Road Signs

BY THE TRAFFIC SYSTEMS RESEARCH DIVISION BUREAU OF PUBLIC ROADS

Reported by [1] Burton W. STEPHENS, Research psychologist, and Richard M. MICHAELS, Chief, Human Factors Research Branch [2]

The reported study was designed to determine general driver ability to steer a vehicle while simultaneously searching for a specific sign and to measure the timesharing basis used in performing the two tasks. Four types of analyses were conducted to determine: (1) the influence of constituents of the tracking task upon performance of the search-and-recognition task, and (2) the influence of components of the search-and-recognition task upon tracking performance. These included testing of the significance of the differences between means of each of the treatments used. Integrated tracking error variances were analyzed, as were the recognition time variances for each of the conditions associated with the dynamic recognition task. In the second kind of analysis, basic to the inquiry was a test of the generalized timesharing hypothesis, which states that there is a trade-off between the two tasks; that very erroneous performance on one of the tasks is highly correlated with relatively accurate performance on the other. The erroneous recognition of signs presented was examined in the third kind of analysis. The fourth kind of analysis was made of control tests that were conducted to: (1) ascertain the possible effects of learning occurring throughout the experiments, and (2) make a comparison of experimental and baseline test conditions in which the tracking tasks and search-and-recognition tasks were presented singly. The tracking data and recognition-time data are separately presented, and the interactions of the two performances are considered together in an examination of the trade-off hypothesis.

Analysis of individual timeshared trials indicated that performance on each of the two tasks is essentially independent. Tests of the specific hypotheses indicate that operator pacing increases as the complexity of the two tasks increases. However, in all comparisons of the independent tasks and their timeshared counterparts, the performance of test operators deteriorated when the tasks were shared. The tests were conducted under highly controlled conditions in the laboratory.

Introduction

MAINTAINING alinement of an automobile on public highways and roads is an essential part of the task of driving and requires the driver to operate the vehicle within somewhat narrow bounds. In addition, he must search for, select, and relate a variety of road signs to his progress and handling of the vehicle. These two tasks require two very different visual tasks that compete for the driver's time and must in some way be evaluated so that minimal variation in steering is achieved. Interactions between the two tasks have not been reported previously by researchers. How these two types of operator performance interact and their effects on highway traffic is not well understood. Yet the efficiency and safety of the highway system depends upon the proficiency of drivers in processing the two tasks. The experiments discussed here provide some basis for the baseline conditions predicted in the research described in this article.

Definitions

The following list contains definitions for some terms used in this article:

Amplitude.—Amplitude is the extreme range of movement obtained by a periodic oscillation translated onto the abscissa of the display (in centimeters when expressed in terms of the display face, or in degrees when expressed in terms of angular extent in relation to location of the eye and the display face).

Coincidence circuit.—A coincidence circuit is an electronic circuit in which an output signal is produced only when each of two or more input pulses occur simultaneously.

Exteroceptive cues.—Exteroceptive cues are information presented to sense organs—visual cues.

Frequency.—Frequency is the number of cycles per second in a periodic vibration.

Kinesthetic cues.—Kinesthetic cues are information (not visible) presented to sensory nerves in the muscles, tendons, and joints.

Line of regard.—The line of regard is an imaginary horizontal straight line through the center of the forehead of the test operator and perpendicular to the plane of a display.

Proprioceptive feedback.—Proprioceptive feedback is an indication of the degree of performance of the muscles, tendons, and joints of the human operator.

Psychomotor response.—Movements of or pertaining to muscular action ensuing directly from a mental process is a psychomotor response.

Objective velocity.—Objective velocity is the first derivative of position with respect to time.

Rho-correlations.—Rho-correlations are rank ordered or ordinal scaled relationships, the degree of which covers a range of plus and minus unity.

Sensory modality.—Sensory modality is a receptor system, such as sight or hearing.

Signing practices.—Signing practices are presently adapted means of presenting road sign information, such as speed limit, route destination, intersection, and curvature.

Sinusoidal waveform or input.—A periodic vibration that rises to its maximum and minimum in equal time by smooth gradation is a sinusoidal waveform or input.

---

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., Jan. 1964.

[2] Members of the Bureau of Public Roads acted as test operators during this study, and members of the Electro-Mechanical Research Branch of the Office of Research and Development aided in developing a facility for Human Factors laboratory research.

Stimuli separation and speed.—The angular displacement between targets or pertinent aspects of displays and the rate of angular change of such a separation is referred to as stimuli separation and speed.

T-test.—The *t*-test is a statistical method used for testing hypotheses comparing the average errors under different conditions.

Time pacing.—Time pacing is the manner in which time is devoted to one task and then another; a pattern that may reveal a regular recurrence.

Visual feedback.—An indication of the degree of performance of visually presented tasks is termed "visual feedback."

Findings

The main findings of the study reported here were:

(1) When a test operator was required to share his time between two tasks, each type of performance was poorer than when he performed only one task.

(2) An increase in the number of message units that appeared in the recognition task did not affect the operator's performance of simulated steering.

(3) The increase in the number of message units that appeared in the recognition task increased the time required to recognize the key word.

(4) As speed of the simulated steering task increased, recognition time decreased.

(5) If a specific message had equal liklihood of appearing or not appearing, recognition time was greater when the key word did not appear.

Background

One-dimensional tracking

Tracking has been assumed to be an analog of the steering task and is so defined in this article. Adams has described it as a paced task where an ". . . externally driven input signal defines an index of desired performance and the operator activates the control system to maintain alignment of the output signal of the control system with the input signal. The discrepancy between the two signals is the error and the operator responds to null the error" (*1*).[3]

Compensatory tracking—simulated steering—provides a signal of any error and requires only a reference and control mechanism. Such a situation is analogous to minimizing deviations from a prescribed path or roadway. Configurations may be displayed on the face of an oscilloscope. Use of a low-frequency sinusoidal wave form provides a geometrically smooth input similar to roadway curvature. The amount of error correction that must be achieved within a relatively stable response period increases in proportion to the frequency of the tracking signal. This requirement may be considered to be an information load. As the frequency input increases, the information load also increases. Consequently, any time that is consumed in the evaluation of other portions of the environment would be expected to lead to less proficient guidance.

Test operators match input frequencies up to 3 or 4 cycles per second (c.p.s.), according to Ellson and Gray (*2*). McRuer and Krendel (*3*) put this figure around 1 c.p.s. Specific controls or display configurations may affect the psychomotor response sufficiently to account for these differences.

In a field situation, amplitude has been shown to affect the operator's error-free performance of guidance. Results of a study were discussed at the National Conference on Driving Simulation in Santa Monica, Calif., late in February 1961, under the title *Simulation Techniques in Automotive Human Factors Research.* It was reported that on a test track having sine wave patterns, drivers reduced their speed in order to effectively track higher amplitude curves. The field study under discussion suggested that increments of angular change remain constant over a time span that probably is associated with the test driver's response time, or an increment of angular displacement of the course target.

Brown (*4*) has shown that many examples involving objects in motion may be seen as a constant proportion of the velocity of angular change, which suggests that a single measure may predict tracking error for a particular control configuration.

Recognition time

Recognition has been defined by Munn (*5*) as the ability ". . . to differentiate beween the familiar and the unfamiliar or what has been experienced and what is new." Recognition time may be defined as the time delay between the presentation of some information display and recognition of it by an observer. In the research reported in this article, search-and-recognition activities were measured. The distinction between simple recognition and the in-motion task discussed here is that the in-motion task was not localized; it had to be pursued, at least briefly. The message had to be searched for, followed, recognized, and finally reported by some action.

In the static state, discriminations have been evaluated from measures of time consumed in making a decision. Considerable evidence supports the assumption that recognition time increases as the task becomes more complex. Classical experiments indicate that reaction time increases as the number of alternate responses increase (*6*, *7*). In 1960, R. D. Desrosiers of the Bureau of Public Roads, in a study of simulated driving, showed that the mean recognition distance of a test operator approaching a sign at a constant speed varied inversely with the number of alternate messages on the sign. On the basis of Desrosiers' laboratory results, an increase in recognition time would be predicted. Although Desrosiers found no significant differences in recognition time in field test data, this lack of significant differences was attributed to the great variance of performance in field situations.

Task sharing

Many studies have been conducted from which information on separate and discrete tasks was organized in the most desirable way and translated to control devices by human operators (*8*). Most of these studies have involved the sharing of tasks between more than one sensory modality.

Briggs and Howell (*9*) have provided data on the interaction of two continuous tracking activities in which the authors considered the effects of speed for each stimulus. These researchers have considered both peripheral and central aspects of the timesharing process. Although more than one source of information was to have been observed during their study, three major differences existed between their study conditions and those reported in this article. First, both tasks required tracking performance that demanded forced pacing in a pattern of alternation between inputs by the operator, unless he neglected one of the two components of the timesharing situation. Second, the spatial separation of targets was fixed for any one experimental condition, reducing search time and also the applicability of such a configuration to the situation in question; that is, information sources used in the Briggs and Howell study were themselves in motion, although the observer was not. Third, neither task required the separate decoding of visual information implicit in the recognition task. Results of all of the studies reviewed indicated a general degradation of performance as task complexity was increased.

In an earlier study conducted at the Institute of Transportation and Traffic Engineering at UCLA (*10*), as many as 15 percent of the drivers were reported to have ". . . made poor selection of routes due to confusion of messages on the signs . . ." or because they ". . . had not seen the signs in time to make proper maneuvers." The driver is faced with a dilemma in negotiating high-speed roadways. A wrong decision in selecting a ramp may cause the driver to go frustrating miles away from his desired destination.

Procedure

On the basis of information developed by other researchers, six hypotheses were formulated for the research reported in this article.

Hypotheses tested

- Tracking error will increase proportionally with the tracking frequency and amplitude.
- Tracking error will increase proportionally with the number of words on the signs (complexity of the search-and-recognition task).
- Recognition time (measured as time from the sequence beginning) will be proportional to the frequency and amplitude of the tracking signal (a sine wave).
- The number of false alarms in recognition will increase proportionally as the frequency and amplitude of the tracking signal is increased.
- Tracking error will not be dependent upon the presence or absence of a specific message on the sign.

[3] References cited by italic numbers in parentheses are listed on p. 88.

• A systematic inverse relationship will exist between tracking error and recognition time if timesharing is determined on an individual trial basis.

The six hypotheses suggested uniform trends but did not predict quantitative values. They were intended to ascertain the relative effects of the concurrent operations that test operators were requested to perform.

Experimental design

A balanced, five-factorial design, in which randomized presentations were made of the different search-and-recognition levels for each tracking condition, was employed. Six test operators were given tracking tests under two amplitude and three frequency conditions. A film that had signs containing two, four, or six messages was used in the tests. The key message, which was PIDFOH, was in half of the presentations. Each of the test conditions was presented to each test operator five times.

Control tests of tracking performance were paired with each of the timeshared (experimental) runs. In the first half of the trials a control test requiring tracking without the filmed presentation was followed by an experimental trial where both tracking and search-and-recognition activities were to be performed. The latter half of the trials for each operator was a reversal; each experimental trial was followed by a control test. Data were collected from 30 trials for each of the tracking control conditions.

Two control trials providing only recognition information were conducted during the first and second halves of the experimental session. No tracking was required.

The tracking task

The test operator was seated, as shown in figure 1, at a console that contained a circular cathode-ray tube (c.r.t.) display, the diameter of which was 12.5 centimeters. The c.r.t. display was situated slightly below the line of vision at a 12-degree tilt to the vertical axis of the room. A low intensity, encompassing brightness of 10^{-3} *mL* surrounded the scope face. The brightness of the tracking signal was approximately 1 *mL*.

The signal was a point of light normally driven horizontally across the face of the c.r.t. and was to be corrected by manipulation of a handcrank. The control mechanism had a 15-centimeter diameter, and a 2.25-kilogram flywheel was attached to smooth the jerking movement of the control system. Two of these cranks were located so that one could be operated easily by the test operator with whichever hand was preferred. The two cranks were mounted on the face of the console at a 45-degree angle to the floor. The crank was adjustable to such a height that the elbow required support by a rubber pad. Hence, rotary control was achieved almost entirely by wrist motion.

The search-and-recognition task

On a rear projection, ground-glass screen a series of motion pictures was presented that contained scenes of a new section of the Interstate Highway near Washington, D.C. The screen was located 8 feet from the seated test operator, and it was vertically alined to the c.r.t. No apparent texture masked the projected images, and the angle of inclination required for a visual shift from one display to another was slight.

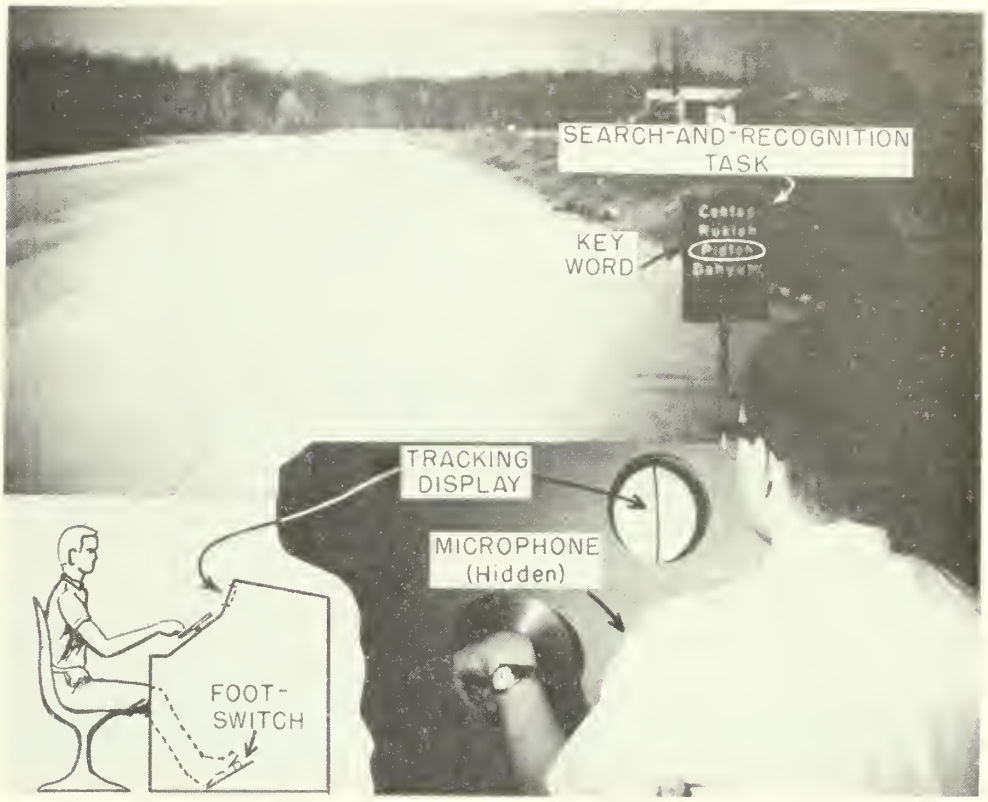

Figure 1.—Test operator seated at driving console, performing tracking tasks and search-and-recognition tasks.

A sign-reading task was shown on the face of the screen. It contained messages composed of nonsense syllables having association values in the range of 27 to 47 percent. Each message unit had six letters, and the lengths of the messages were matched, as were the contours of the letters in the message units. The apparent size of the signs was the same as those on an actual highway. That is, the visual angle opposite the sign at the operator's eye level was matched for the time from the sign that corresponded to the distance of an actual vehicle traveling at constant speed. Each approach scene was projected at a rate of 24 frames per second, and lasted 16 seconds, including 1 second of film that simulated passing and receding of the sign.

Test Method Operators

To conduct the tests developed to evaluate the hypotheses previously listed in this article, equipment was set up in the Bureau of Public Roads laboratory and operators were selected. The general relationship of the task configuration to the response devices is shown in figure 2. Each of these blocks in its inter-

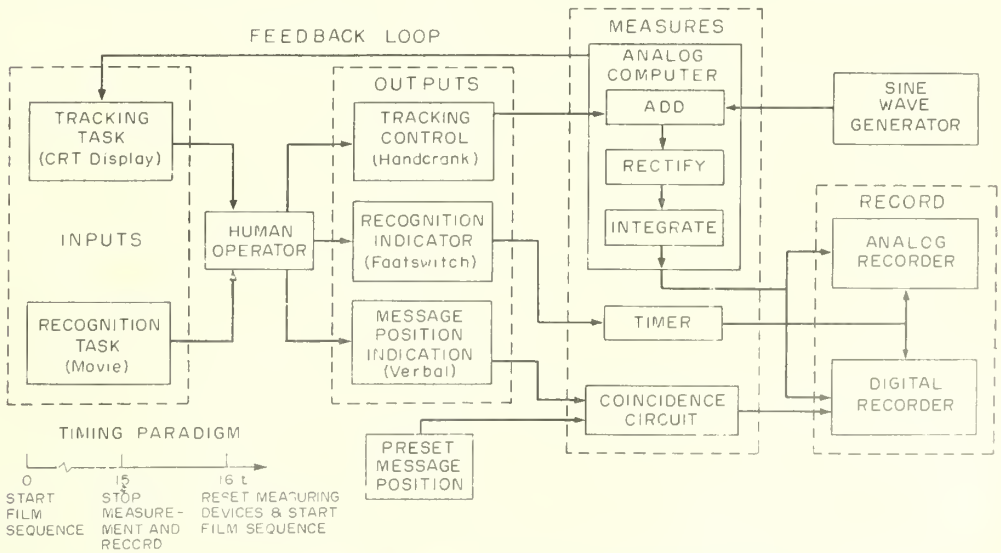

Figure 2.—Timesharing simulation system block diagram.

acting role is discussed in the following paragraphs. In addition to the presentation and response systems, timing, measurement, and recording systems for the experiment are also discussed. The relative positions of the displays and control devices are shown in figure 1.

Six test operators from the staff of the Bureau of Public Roads, Office of Research and Development, were selected at random. The four males and two females, whose ages ranged from 21 to 45 years, reported no irregularity of vision or psychomotor impairment, and all had had several years of driving experience.

Prior to the experimental session, the test operators received approximately 4 hours of training in tracking and search and recognition. The film for the recognition control tests was also used for training. Training for all but the last operator was given the day preceding the experiment. The experiments were divided into two 3-hour sessions. Each of the 14 trials lasted 8 minutes, and rest periods between trials lasted for approximately 8 minutes.

During the experimental runs, the test operators were requested to simultaneously track—keep the dot on the c.r.t. display centered on a vertical black line dividing the scope face—and to indicate when the word PIDFOH could be recognized on the motion picture display. Operators pressed the foot switch as soon as they recognized PIDFOH or as soon as they recognized that the key word was not shown. Then, immediately, they verbally gave the position of the key word by reporting the numerical position—1 through 6—or zero when PIDFOH did not appear on the sign. Operators were not instructed to pace the task in any particular manner, but merely to perform as well as possible throughout each trial. They were told to indicate recognition of the key word as soon as possible, without guessing. Otherwise, the experimental situation was relatively unstructured.

Prior to the photographing of the highway section, portable sign standards were erected. Their green backgrounds nearly reproduced the color of the present standards used for destination signs on The National System of Interstate and Defense Highways. During photographic sessions, interchangeable messages were placed on the standard backgrounds, and the signs were erected so that the brightness was approximately equal for all. The camera was mounted near the head position of the driver. Then motion pictures were taken that, from the operator's view, would give the test operator the feeling of driving on a roadway.

During photography, four standards were placed 1,000 feet apart. One of 30 sign configurations, selected at random, was assigned to each location. Each sign contained a combination of two, four, or six message units, with or without the key message PIDFOH, and the position of the key message was also assigned at random to one of the available positions.

The developed pictures were duplicated so that seven completely different edited versions were available for the experiment. Each film had the 30 scenes of the standardized signs in a different order.

Timing

The timing system provided sampling of performance on the two timeshared tasks. The timing pattern is shown in figure 2. Timing of the two tasks was synchronized by employing the film as a common time base. A small hole was cut in the corner of frames that would be shown 15 seconds and then 1 second part. Light from the projector source energized a photoresistor in the upper corner of the screen. The test operator seated on the other side of the ground glass screen could not see the flashes of light from the projector and, hence, saw only what appeared to be a continuous roadway.

The photoresistor pulsed a flip-flop mechanization that alternately: (1) reset the measurement devices to a deenergized state, followed by an activated state of measurement, and (2) stopped the measurements and digitally recorded performances on each of the test operator's tasks.

Measurement

Measurements of rectified integrated tracking error, time from the sequence start, when test operators indicated recognition associated with the sign-reading task, and correctness of verbal response to the key word on the highway sign were recorded digitally. These measures were used as the dependent variables for determining the tenability of the timesharing hypotheses. Analog records of tracking performance and a separate indication of recognition time were also collected.

The system of measurement of cumulative tracking error is shown in figure 3. This error measurement system is essentially the one developed by Gain and Fitts (*11*) for a series of tracking studies. In this system, one of the cranks was attached to a potentiometer for measuring and controlling a d.c. voltage. This voltage in turn, was added to a low-frequency sine wave, and the algebraic sum (shown on the cathode-ray display) was multiplied by a constant value empirically derived for minimal error of computation, rectified, and integrated. An output pulse from the flip-flop actuated a double-pole, double-throw relay between the rectifier output and the integrator at the termination of each 15-second sampled sequence. This relay in turn energized a delay sample circuit associated with a d.c. digital voltmeter. The voltmeter in turn issued a print command to a digital recorder. Total system error measured during any one day was less than 1 percent.

The sampled sequence initiation pulse from the flip-flop actuated another double-pole, double-throw relay that issued a short pulse to the clamping circuit associated with the integrator, thereby zeroing the unit and permitting an accurate cumulative error score to develop. Time from the end of the sampled sequence to recognition by the test operator was measured by the second set of contacts on this pulsed relay. A positive pulse of 80 volts started the count of an electronic timer. A stop pulse was initiated by either the test operator's pressure on a foot switch or by the one set of relay contacts associated with the stop phase of the flip-flop.

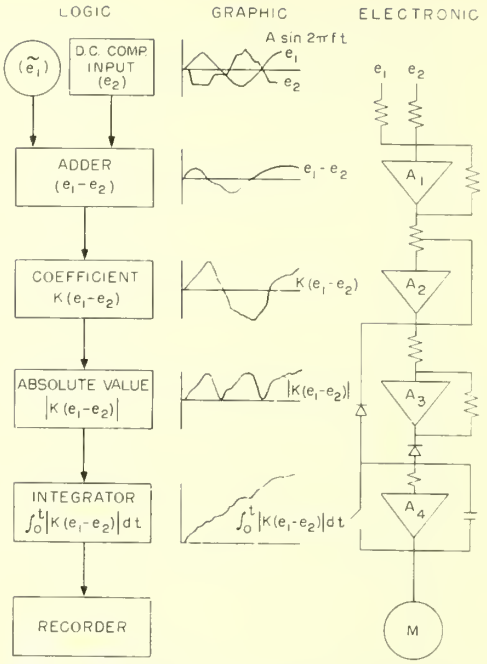

Figure 3. Method for recording tracking error—logically, graphically, and electronically.

Verbal indication of the position of the key word on the search-and-recognition display was measured for correctness of response. At the beginning of each trial, for each sequence, the experimenter set into a series circuit the position associated with the key word. The test operator's verbal response was recorded when he pressed the second switch in the pair of cascaded switches. If the position reported was the same as that preset by the experimenter, the digital recorder printed an indication of position correctness.

Discussion

Interactions between the independent aspects of tracking and search-and-recognition activities have not been reported on by previous researchers. Although results of studies on timesharing between different sensory modalities and two-dimensional tracking have indicated general degradation of performances beyond certain input loads, consideration of tasks having different, basic functional requirements has been lacking. Information on an investigation into the interaction processes between these two tasks is reported in this article. Changes in responses, when tracking and search-and-recognition performances were measured in a nontimesharing role, have also been compared.

Tracking Performance

The five factors of tracking performance and their interactions, as used in the experiments, are summarized in table 1. These data reflect analysis of presentations when test operators were required to perform tracking and search-and-recognition tasks concurrently. Inferences drawn from these data, therefore, do not reflect the effects of only the tracking

Table 1.—Cumulative tracking errors: Summary of analysis of variance

Factor	Sum of squares	*df*	Mean sum of squares	F, ratio
Frequency	5,582.52	2	2,791.26	[1]493.15
Amplitude	4,593.27	1	4,593.27	[1]811.53
Messages, number	22.58	2	11.29	1.99
Presence	59.52	1	59.52	[1]10.52
Test operator	2,481.67	5	496.33	[1]87.69
FxA	2,244.01	2	1,122.00	[1]198.23
FxM	33.85	4	8.46	1.49
FxP	14.01	2	7.00	1.24
FxT	490.69	10	49.07	[1]8.67
AxM	1.06	2	0.53	0.09
AxP	12.97	1	12.97	2.29
AxT	635.52	5	127.10	[1]22.46
MxP	26.58	2	13.29	2.35
MxT	35.39	10	3.54	0.62
PxT	19.82	5	3.96	0.70
FxAxM	22.10	4	5.52	0.98
FxAxP	13.69	2	6.84	1.21
FxAxT	509.41	10	50.94	[1]9.00
FxMxP	60.59	4	15.15	2.68
FxMxT	124.93	20	6.25	1.10
FxPxT	50.21	10	5.02	0.87
AxMxP	12.76	2	6.38	1.13
AxMxT	49.89	10	4.99	0.88
AxPxT	25.47	5	5.09	0.89
MxPxT	87.22	10	8.72	1.54
FxAxMxP	54.24	4	13.56	2.40
FxAxMxT	108.15	20	5.41	0.96
FxAxPxT	60.65	10	6.06	1.07
FxMxPxT	146.28	20	7.33	1.30
AxMxPxT	17.23	10	1.72	0.30
FxAxMxPxT	157.80	20	7.89	1.39
Residual	4,894.44	864	5.66	------
TOTAL	22,676.20	1,079	--------	------

[1] Significant with probability less than 0.01.

attribute but also the influence of attributes of the search-and-recognition task. Data in table 1 show that on the basis of frequency, amplitude, and presence of the key word operators have demonstrated differential effects greater than can be expected by chance 1 percent of the time. Further, only the combined interactions involving frequency, amplitude, and operators are significant.

Tracking error as a function of the significant variables of frequency, amplitude, and presence of the key word is shown in figure 4. The data for all test operators have been combined. The differential effects of tracking error compared to the control condition (tracking without the timeshared task) are clearly shown by the information in figure 5. These data showed the relative magnitude of differences and illustrated the degredation caused by combining the two tasks. In both the low- and high-amplitude tests, a clear decrement in proficiency occurred when the operators were confronted with the two tasks. The error score in the dual task increased overall about 80 percent; but the differences between the presence and absence of a key word were separated by only a small but reliable amount—about 15 percent.

Using a method of matched test conditions for all test operators, both presence and absence of the key word and all sign complexities, a comparison by means of student's *t*-test of the lowest frequency-amplitude condition, with and without the interpolated task, yields $t=3.92$, which is significant at the 0.01 level. Therefore, tracking performance for the lowest average velocity condition was considered to be superior to the comparable timeshared condition.

Search-and-recognition time

Test operators' differences in recognition time are also significant, as were measures of tracking performance. A summary is shown in table 2 for recognition-time conditions when the key message was on the sign. Of the two components of the tracking task, only frequency appears to have differentially affected performance on the dynamic recognition task. In figure 6, recognition time data is shown for each of the tracking frequencies. The effects of task sharing may be seen. The curve in this figure is only for illustration because operator differences influenced such a plot. The time required to recognize the key message on the sign was greatly increased when tracking was used as a secondary task.

The lack of interactions involving the number of message units on a sign indicated that test operators responded in essentially the same manner to different numbers of message units on the signs. The differences between two, four, and six words reflected a steady increase in recognition time as the number of message units increased. The time increase between the two- and six-message signs rose only about 4.4 percent, or slightly less than one-half second. The range of increase on the timesharing tests was from approximately one-half second to 2 seconds beyond the recognition time for the control tests when the film was shown without the displayed tracking task.

The means of both the control and experimental conditions are shown in figure 7. For the control conditions, a dip is shown at four message units per sign. This effect does not parallel the general trend of the data from experimental tests. The time differences were significant between two and four message units at the 0.05 level but failed to reach significance between four and six units.

Search-and-recognition performance was tested to learn whether there were any changes between the first and second halves of the experiments. Using a *t*-test for matched test conditions, the mean recognition times for each operator were compared for the 5th and 10th trials. No statistical difference at the 0.05 level was obtained.

The errors in judgment were determined from the responses of the test operators to the position of the key word on the signs. The error scores are listed in table 3 by percentage of misses and by false alarms—the error rates incurred by each of the operators when they reported the key word in wrong positions or in a position when it was not present (false alarms). These rates were sufficiently low so that no test was required to discriminate between guessing and nonguessing. The total number of errors was well below the 5 percent criterion imposed; therefore, the erroneous responses were not analyzed further.

Timesharing

If operators gave their best performance on one of the two imposed tasks, they should have given performances that were very negatively correlated with each other. Samples of performance were taken at random for

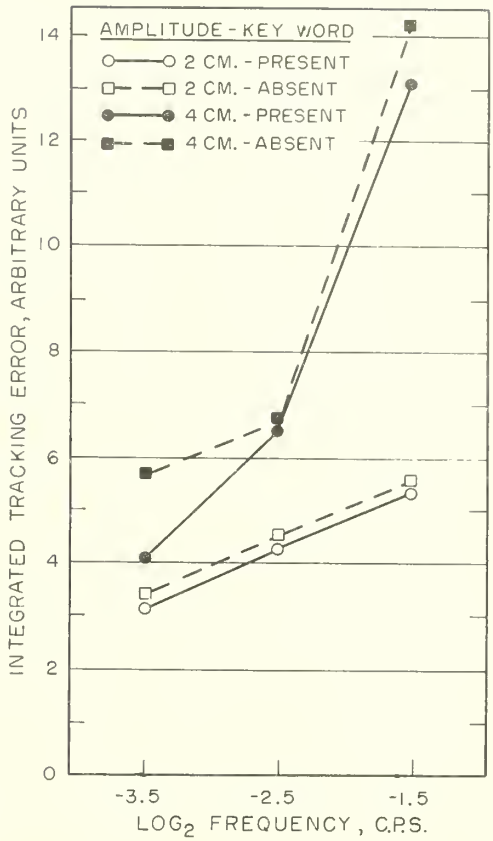

Figure 4.—Tracking error data showing relative effects of frequency, amplitude, and presence of the key word.

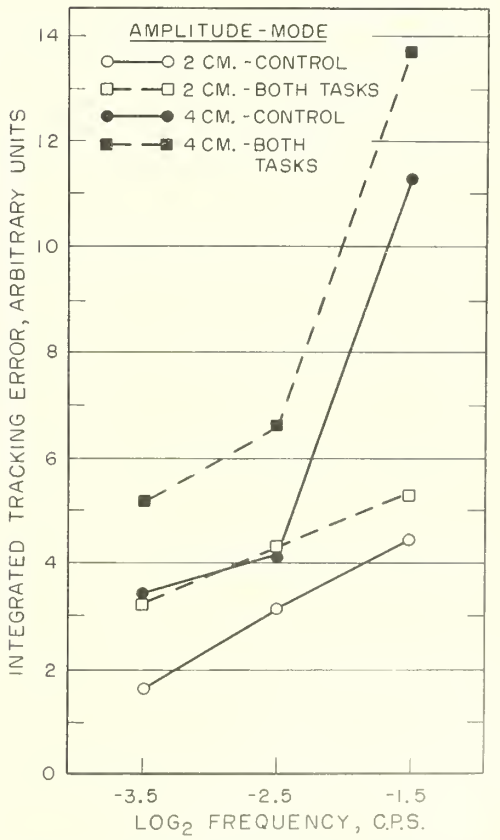

Figure 5.—Tracking error data showing relative effects of the sign reading task.

Table 2.—Recognition time: Summary of analysis of variance

Factor	Sum of squares	*df*	Mean sum of squares	F, ratio
Frequency	9.53	2	4.76	[1] 4.81
Amplitude	3.82	1	3.82	3.86
Messages, number	20.61	2	10.30	[1] 10.40
Test operator	1,062.65	5	212.53	[1] 214.68
FxA	5.84	2	2.92	2.95
FxM	2.29	4	0.57	0.58
FxT	28.22	10	2.82	[1] 2.85
AxM	0.11	2	0.06	0.06
AxT	5.45	5	1.09	1.10
MxT	17.14	10	1.71	1.73
FxAxM	4.80	4	1.20	1.21
FxAxT	28.12	10	2.81	[1] 2.84
FxMxT	27.48	20	1.37	1.38
AxMxT	11.13	10	1.11	1.12
FxAxMxT	28.89	20	1.44	1.45
Residual	1,256.08	107	---	---
Error	428.47	432	0.99	---
TOTAL	1,684.55	539	---	---

[1] Significant with probability less than 0.01

timesharing conditions of individual operators. Twenty samples were subjected to rho-correlations. Only one correlation was significant at the 0.05 level. Therefore, the hypothesis that a significant correlation existed between the two timeshared performances must be rejected because the number of significant correlations did not exceed those expected by chance alone.

The conclusion that no significant correlation existed between the timeshared performance was derived from an analysis of the pattern of response, the number and dispersion of errors of tracking and search-and-recognition performance. The fact that the number of messages on the signs had no effect on tracking performance suggested that the overall time required by the dynamic recognition task was not affected differentially by the number of messages. Yet the number did contribute to an increase in recognition time. Such a paradox suggests that either: (1) The differences in recognition time, although significantly different, were so small that they did not affect tracking performance; that is, the variance associated with other factors was relatively large; or (2) the test operator quickened his alteration response to or from the tracking task and decreased time spent in the search-and-recognition task.

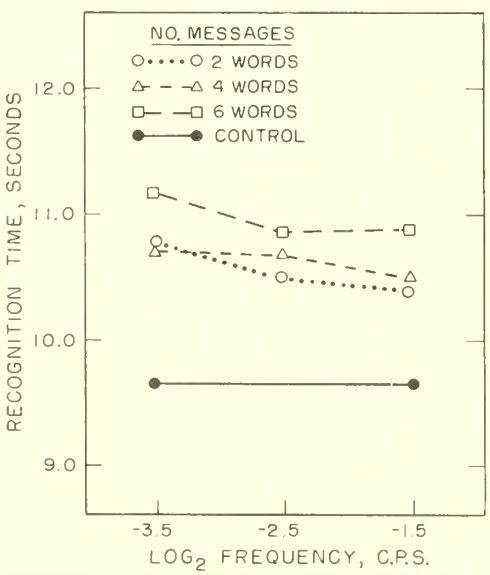

Figure 6.—Recognition time data showing effects of timesharing with tracking task.

Differences in the differential error between timeshared tests and control tests involving tracking alone are not great for each of the frequency-amplitude combinations. The trend, however, is toward smaller differentials, rather than greater ones, as the tracking frequency is raised. If a constant differential between each control and experimental comparison is assumed, then the same increment of error can be attributed to each of the frequencies.

Although no difference exists between the integration of sine waves of differing frequencies (unless the integration time is very short), the lack of visual feedback for 1 or 2 seconds becomes increasingly important as the frequency is increased. Gottsdanker (*12*) has indicated that essentially the same processes operate in anticipating a course under visual and visually deprived conditions. Test operators, however, showed more variable and less accurate performance when they were visually deprived. The operator maintained some proprioceptive feedback and was able to anticipate the course, but this ability decreased as frequency was increased.

Briggs and Howell imply that when tasks to be performed are spatially separated the operator must discretely sample these tasks. When more than one task competes for the operator's efforts, then the sampling rate is paced according to the objective velocity of the stimuli of the tasks, but often this rate is modified by some subjective criterion. A decrement in performance shows a lack of visual error feedback or, expressed differently, a reduction of the sampling rate of the error signal by the test operator. If the rate of increase is less than would be expected by reduced feedback information, then the operator must increase his sampling rate; this has an associated effect on some other aspect of behavior. This may be presumed to be true only if the operator is responding well within the limits of his capacity to assimilate information.

Recognition or Tracking Error

If equal time is spent on the search-and-recognition task and is independent of the tracking task, then the analysis of variance of recognition times should show no significant differences for tracking frequency, amplitude, or their interactions; that is, recognition times would not be increased or decreased because of the addition of the secondary task (but, as stated earlier, the average recognition times did decrease as the frequency of the tracking task was increased). Furthermore, if an increment in tracking error exists in the timesharing condition, then the size of such an increment should increase as a function of increased frequency. If the reverse were obtained, then it might be presumed that a time reduction in recognition time had been used in minimizing tracking error. The test data clearly substantiated these presumptions. Although the number of messages displayed to operators had no effect upon their tracking performance, frequency and the complex interactions of frequency, amplitude, and test operator substantially affected recognition time. A reasonable inference is that, as frequency (a component of angular change of the target) was increased, the rate of sampling of the course increased and search-and-recognition behavior of the operator was consequently altered. Comparable to the field situation, in which the steering task may be considered in the same context as tracking, the operator's search-and-recognition behavior will decline as sign reading becomes more complex but will actually improve as traveling speed is increased.

Table 3.—Erroneous response percentages, by test operators

Test operator	Error	False alarm	Total erroneous response
	Percent	*Percent*	*Percent*
1	0.8	1.7	1.2
2	5.8	2.5	4.2
3	2.5	0.8	1.7
4	1.7	0.8	1.2
5	0.8	0.0	0.4
6	0.8	0.8	0.8
Average	2.1	1.1	1.6

The effects on tracking performance when the key word did not appear on the sign suggest that expectancy played some part in diverting the attention of test operators from the simulated steering. The significant increase in tracking error leads to the conclusion that more time was spent in attempting to find the missing key word.

Cues in Nontimeshared Recognition Trials

In the comparison of control and experimental data, a decrease was noted in recognition time associated with increased numbers

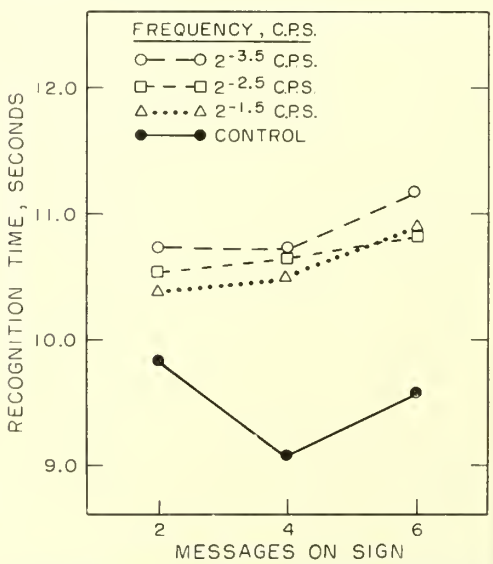

Figure 7.—Recognition time data showing effects of number of words on signs and tracking frequency.

of messages in the search-and-recognition activity when no timesharing was required. It is most likely that, because of using the control film for training, disproportionate learning was associated with the four message signs. There was no way of directly testing this hypothesis. Close observation of these films, however, revealed considerable activity in the periphery of the scenes during photography of the four message signs. When test operators were required to view only the film (which had been repeatedly shown during training), it is likely that such peripheral cues were noticed and used. This most likely explains the U-shaped curve associated with the control tests, shown in figure 7.

Spatial Separation of Information

Although care was taken to present a reasonable projection of the roadway (rates of change of objects in the simulated view of the road being kept comparable to those in the field), accuracy of detail in the films used required much larger messages for recognition. This, in effect, also meant that the lateral movement of objects across the viewed field was much further from the line of regard in the laboratory situation than on an actual highway. Concurrently, the angular velocity of the signs was much higher at the time of recognition. This constraint meant that the message, when detected, was displayed about 10 degrees from the line of regard. In the field, however, previous tests under comparable speed conditions showed recognition of signs of the type used in this study to be at about 400 feet. At the time of recognition, the displacement of the sign was about 2 degrees from the line of regard, which is within the foveal region of the eye. The shift of the point of fixation for the field situation was very small, thereby reducing the transition time between the two information sources: visual steering feedback and the messages on the signs.

Shifting cues

Control devices used to respond to repetitive inputs permitted development of some reliable cues for the test operator in the simulated steering task. Because the compensatory tracking signal produced high redundancy in the experiment described, the error, incurred during periods when the operator directed his attention toward the signs, was probably minimized. In tasks dependent upon both visual and proprioceptive cues, Bahrick and Shelly (*13*) determined that during prolonged trials a gradual change occurred from visible cues to proprioceptive control. So, indeed, there may have been a reduction in the errors scored under these high redundancy conditions. However, the use of a balanced design in analysis of errors scored reduced the influence of this factor in the findings reported here.

Frequency-Amplitude

Frequency and amplitude are recognized in this article as aspects of the same phenomena—rate of angular change of the source of the tracking target stimulus. A report by Bowen and Chernikoff (*14*) contains a hypothesis concerning the cumulative tracking error of test operators; they suggest that an error estimate may be determined by the frequency-amplitude product,

$$E = k(fa).$$

Their tests of this relationship yielded good fits of the data. Although this relationship did not take into account the gain of the specific control used by operators, significant trade-off between these two aspects of the tracking task provides the reason as to why each should have had pronounced differential effects and as to why an interaction should have existed between them.

Bowen and Chernikoff showed that equal frequency-amplitude combinations should yield equal tracking error. This point is not germane to the central topic of this article, but making comparisons of conditions having equal average velocities is essential to determining the applicability of the principle in the body of the research reported. So as to not confound the argument by possible interactions not shown by previous analysis, matched groups of all test conditions: operator, presence or absence of the key word, and complexity of tasks were compared for the equal frequency-amplitude combinations. Where,

$f_1 = 2^{-3.5}$ c.p.s.,
$f_2 = 2^{-2.5}$ c.p.s.,
$f_3 = 2^{-1.5}$ c.p.s.,
$a_1 = 2$ cm.,
$a_2 = 4$ cm.,

and E is the error associated with conditions dictated by the subscripts, the following listed hypotheses were tested by use of the t-test. The corresponding probabilities of the differences exceeding t are given:

Hypotheses		*Statements*
	H1,	$E_{f_1 a_2} = E_{f_2 a_1}$, $0.3 < p < 0.4$;
but,	H3,	$E_{f_1 a_1} = E_{f_1 a_2}$, $p < 0.01$;
and,	H2,	$E_{f_2 a_2} = E_{f_3 a_1}$, $0.5 < p < 0.6$;
but,	H4,	$E_{f_2 a_2} = E_{f_2 a_2}$, $0.05 < p < 0.10$.

On the basis of the preceding analyses, the frequency-amplitude equality concept can be considered to be valid. The differences between the equal fa products were not significant. However, when the cumulative error was compared for matched groups for like frequencies but different amplitudes, differences were significant or were near the critical level of accepted difference.

Angular Velocity Stimulus

In addition to Bowen and Cherikoff's treatment of the frequency-amplitude combination as a predictor of tracking error, Brown has classified these components as those that may be translated into angular velocity, a primary stimulus to the human operator. Such a concept permits the translation of vehicular speed and road curvature into these terms. The stimuli used ranged in maximum angular velocities from 0.74 to 5.37 degrees per second For a curve having a radius of 1,330 feet with a superelevation of 5 percent, AASHO design standards indicate an angular velocity range of 2.59 degrees per second for a vehicle entering the curve at 60 m.p.h., down to 1.50 degrees per second for a vehicle entering at 30 m.p.h. On such curves the driver does not in theory need to make more than an initial correction, whereas test operators in this study were negotiating a very crooked road.

Although the differences between tracking error for equal frequency-amplitude combinations were not greater than could be expected by chance, the two amplitude conditions exhibited relative differences that are worthy of note. Inspection of the configurations confronting the test operators showed at least two types of differences between the conditions. The first pertained to the angular velocity of the control conditions: The average velocities for test differed almost 30 percent, but the maximum velocities did not differ; in the other test, the maximum velocities differed by almost 10 percent, but the average differed only a few percent.

The Right Decision

It is especially critical to the driver who is unfamiliar with a specific course to make the right decision. He may attempt to make the best of the situation so that he has more time in which to reach a decision—he may reduce speed. He may direct his attention primarily to the roadway from external visual sources, such as signs. It is conceivable that his control, particularly lateral control, will decrease appreciably.

The suggestion that substantial research is needed about locations where the driver must make decisions concerning alternate courses is supported by the evidence from the study discussed in this article of the interaction effects of steering and search-and-recognition activities, together with the report of the ITTE. Congestion along the highway, such as at ramps, does not explain the many accidents in these areas. Mullins and Keese (*15*) report that more than 20 percent of accidents in freeway sections studied occurred at ramps where signs or oncoming traffic must be searched out, located, recognized, and acted upon. If recognition is to occur in time for a decision, then scanning time on driver's position on the roadway must be decreased (but this was not indicated by data collected in this research) or the angular velocity of the roadway must be decreased. This may be accomplished by a decrease in driving speed, a factor probably associated with accident rates (*16*). Where speed variability is high on high-speed roadways, there is also considerable evidence of a reduction in road capacity (*17*).

The issues to consider are alternate methods of presenting information to the operator so that he is not overburdened, and determination of ways that will permit the driver to

maintain speed for such decision points along the highway. Finally, where a message is definitely expected, an indication of the proper direction should be on the sign, otherwise more time will be given to the sign reading task than to maintaining steering control.

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Evaluation of Laboratory Vane Shear Test

BY THE MATERIALS DIVISION
BUREAU OF PUBLIC ROADS

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The laboratory vane shear device produces shear strength measurements that are reliable, according to the research results reported in this article. The accurate determination of the shear strength of soft foundation soils is an important factor in the design of modern highways. Consequently, the Bureau of Public Roads is conducting a study of laboratory shear strength tests and devices to determine the strength parameters measured by the different laboratory tests and to evaluate the usefulness of such tests in relation to highway design. The test discussed in this article covers only one facet of the study. The results obtained on the evaluation of the laboratory vane shear test show that this device provides reliable values of shear strength. This, coupled with the ease of testing and the relatively small degree of disturbance of the specimen, makes the vane shear test a desirable tool, especially for use on soft, sensitive soils.

Introduction

WITH THE present expansion of the highway construction program, the highway engineer is frequently required to locate and build highways and particularly interchanges over poor, submarginal lands, sometimes even swamplands. One problem in designing interchange embankments over soft soils is the determination of the initial shear strength of the foundation soil. This shear strength may be estimated from the results of the standard penetration test, obtained during subsurface investigations, or measured directly by the field vane test. In general, however, the initial shear strength of the foundation soil is based upon laboratory unconfined compression tests of undisturbed samples. When the undisturbed samples are very sensitive, or so weak they fail under their own weight, another type of test is needed.

The Materials Research Division of the Bureau of Public Roads has purchased a laboratory vane shear device (figure 1) for testing very soft soils directly in the sampling tubes. The test consists of inserting the vane into the sample and measuring the resistance to rotation. The shear strength can then be calculated from this resistance to rotation. This device was evaluated by comparing the shear strength result obtained thereby with the shear strength result obtained from the unconfined compression test.

In initial tests, the laboratory vane shear test and the unconfined compression shear test did not yield the same shear strengths for identical remolded specimens of soil. In an attempt to explain this difference, additional tests were run on both undisturbed and remolded soils. The remolded soils were studied for the effects of aging of the specimens before their being tested, the rate of shear, and the vane size.

A few researchers have compared the shear strength obtained from tests made with the field vane, the laboratory vane (undisturbed sample left in the tube when tested), and the undisturbed, unconfined compression tests (*1*, *2*).[3] Generally the field vane test results showed the greatest shear strength, the unconfined compression test results showed the least strength, and the results of the laboratory vane test showed a strength between the other results. Gray (*1*) believed that for the sensitive clays he tested, the difference was caused by the disturbance of the soils when the samples were obtained and when the unconfined compression test specimens were prepared. Results of the consolidated undrained triaxial shear tests made by Fenski (*2*) were close to results of the field vane test. Therefore, he concluded that the difference between the results of the field vane test and the unconfined compression test was caused by the different stress conditions created when the soil was removed from the ground.

Summary

The results of the study reported in this article are as follows:

Laboratory vane shear and unconfined compression tests on a variety of fine grained soils, molded into test specimens at moisture contents between the plastic and liquid limits, showed that the ratio of the vane shear strength to one-half the unconfined compressive strength was constant for a given soil. This vane-*uc* ratio varied from soil to soil, ranging from 0.6:1 to 1.4:1. Soils having plasticity indexes of less than 14 had vane shear strengths less than the unconfined compressive shear strengths, and soils having plasticity indexes greater than 14 had vane shear strengths greater than the unconfined compressive shear strengths. Possible reasons for this difference in shear strengths are:

- A remolded soil having a high plasticity index probably has nonuniform structure. As the test specimen in the vane test failed along a fixed surface and the test specimen in the unconfined compression test failed in the weakest area, nonuniformity of structure, caused the vane strength to be greater. The vane-*uc* ratios of the undisturbed soil increased from 0.9:1 before being remolded to 1.20:1 after being remolded. Also, the vane-*uc* ratio decreased as the age of the specimen increased. It is probable that the structure of undisturbed samples and specimens aged after their being remolded was more uniform than the structure of specimens immediately after they had been remolded, thus causing the lower vane-*uc* ratio.

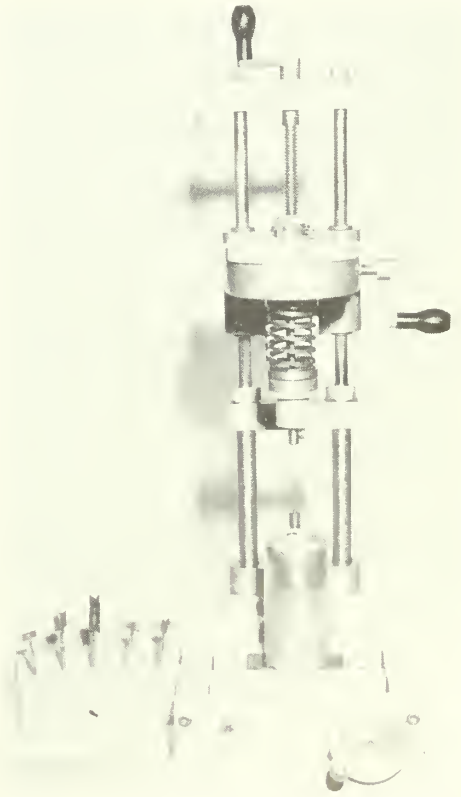

Figure 1.—Laboratory vane shear apparatus.

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., Jan. 1964.

[2] Comments on the tests reported were made by Professor Nyal E. Wilson, Department of Civil Engineering and Engineering Mechanics of McMaster University, Hamilton, Ontario, Canada, and are reproduced at the end of this article. Comments of the authors on Professor Wilson's remarks are also included.

[3] Numbers in parentheses indicate references listed on p. 93.

Table 1.—*Summary of soil tests and soil properties*

Group and type of test	Test	Ratio of vane shear to one-half *uc* strength	Ratio of penetrometer shear to one-half *uc* strength	Samples tested	Moisture content, *w*	Dry density	Basic soil properties							
							Liquid limit	Plastic limit	Plasticity index	Clay, <0.005 mm.	Silt 0.074–0.005 mm.	Sand 2.0–0.074 mm.	Classification	
													AASHO	USDA texture
A:	*No.*			*No.*	*Percent*	*P.c.f.*								
Determination of vane-*uc* ratio (remolded soils).	1a	1.0:1	1.5:1	54	37.0 to 44.0	73.0 to 80.6	51	30	21	44	49	7	A-7-5(15)	Silty clay loam.
	2a	1.4:1	1.5:1	22	20.3 to 35.5	83.2 to 97.9	48	20	28	37	23	40	A-7-6(13)	Clay loam.
	3a	0.8:1	2.0:1	6	33.6 to 35.1	80.7 to 82.9	41	35	6	4	33	63	A-4(4)	Sandy loam.
	3b	1.3:1	1.6:1	3	35.9 to 36.0	89.1 to 89.8	36	28	8	22	34	44	A-7-6(12)	Loam.
	5	1.4:1	2.1:1	3	47.8 to 47.9	71.7 to 71.8	49	33	16	83	10	7	A-7-5(20)	Clay.
	6	1.25:1	1.5:1	3	64.0 to 65.0	59.6 to 61.1	88	41	47	92	7	1	A-7-5(20)	Clay.
	7	0.6:1	2.1:1	3	24.7 to 24.8	98.0 to 100.7	89	34	55	77	12	11	A-2-4(0)	Clay.
	8a			8			30	24	6	15	18	67		Sandy loam.
		1.3:1	1.7:1		33.9 to 34.0	85.9 to 87.0							A-7-6(19)	
	9a			3			55	25	30	83	13	4		Clay.
B:		0.9:1	1.1:1		46.2 to 47.9	109.7 to 112.0							A-7-6(14)	
Determination of vane-*uc* ratio (undisturbed soil).	10a			5			46	24	22	46	52	2		Silty clay loam.
		1.2:1	2.0:1		44.8 to 46.3	110.4 to 118.5							A-7-6(14)	
	10b			7			46	24	22	46	52	2		Silty clay loam.
C:		1.4:1			25.0 to 31.6	89.7 to 98.7							A-7-6(10)	
Rate of shear.	2f	0.5:1 to 1.0:1	2.3:1	10	22.1 to 22.6	101.9 to 102.7	41	19	22	37	23	40	A-2-4(0)	Clay loam.
	8b			8			30	24	6	15	18	67		Sandy loam.
D:		0.3:1 to 1.4:1			26.2 to 26.7	95.5 to 96.3							A-7-6(10)	
Effect of age.	2d	1.4:1 to 1.5:1		6	24.9 to 25.8	96.9 to 97.5	41	19	22	37	23	40	A-7-6(10)	Clay loam.
	2e	1.2:1 to 1.4:1		9	32.0 to 33.0	87.7 to 87.9	41	19	22	37	23	40	A-7-6(19)	Clay loam.
	9b			11			55	25	30	83	13	4		Clay.
E:		1.1:1	1.7:1		38.1 to 38.9	78.2 to 82.9							A-7-5(8)	
Effect of vane size.	1c	1.3:1	1.3:1	18	24.9 to 34.8	84.3 to 98.9	51	30	21	44	49	7	A-7-6(10)	Silty clay.
	2b			54			41	19	22	37	23	40		Clay loam.
F:		1.3:1 to 2.7:1	1.3:1 to 2.2:1		19.0 to 21.4	102.3 to 107.3							A-7-6(10)	
Effect of testing procedure.	2e			8			41	19	22	37	23	40		Clay loam.

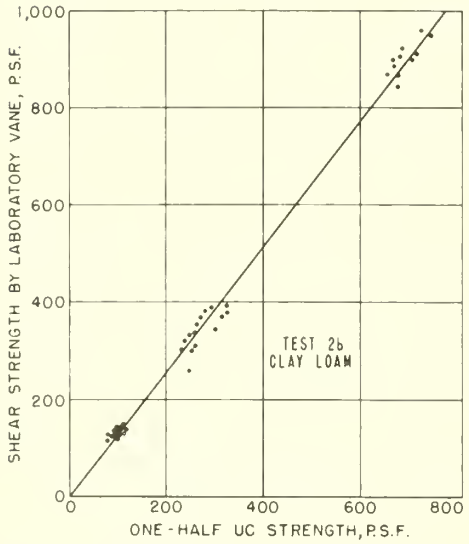

Figure 2.—Vane shear versus one-half unconfined compressive strength.

• A soil having a low plasticity index probably has an intergranular friction force that is mobilized in the unconfined compression test, but not in the vane shear test. This causes low vane-*uc* ratios for such soils.

The buildup of pore-water pressure in the test was studied indirectly by the effect of rate of shearing on the vane-*uc* ratio, but the results were inconclusive. However, increasing the rate of rotation of the vane for a soil having a low plasticity index did cause an increase in the shear strength. As the soil tested was dilatant, negative pore-water stresses were probably induced during the shear deformation. This negative pore-water pressure contributed to the increase in shear strength. No such effect was noted for clays. The length and the number of blades made little difference on the results of the vane test for the soils tested.

The difference noted by other researchers between the unconfined compressive shear and the vane shear strengths may have been caused by differences in the modes of failure rather than in the actual shear strength of the soil.

The shear strength values obtained with the laboratory vane shear device appear to be reliable. The ease of testing and the relatively small degree of disturbance to the specimens make this a desirable test, especially for soft, sensitive soils.

Vane Shear Apparatus

The vane shear apparatus used for the study reported here (fig. 1) is equipped to measure both torque and rotation of the vane in degrees. Torque is applied to the vane through a calibrated spring by rotating the crank handle. The base of the apparatus was replaced by a clamp to hold the sampling tubes. The vane normally used has four blades, each one-half inch in height and one-fourth inch in width.

Calculation of Shear Stress

The surface of rupture and the possibility of progressive failure for the vane test has been studied for sand and clay by Swedish researchers (*3*). Their study was conducted by placing wetted tissue paper, which had a pattern marked on it, on the surface of the samples. The distortion of the pattern was observed as the vane test progressed. It was concluded that, for the sand and clay tested, the surface of rupture was a circular cylinder of which the diameter (D) equaled that of the vane, and any progressive character of failure was slight and did not appreciably affect the test results.

When the vane shear strength, S, was calculated in the study reported here, it was assumed that the failure surface was the circular cylinder of revolution created by rotating the vane. The shear stress was assumed to be constant along the vertical surface and to have a linear distribution on the ends ranging from zero at the center to maximum at the edge. The resultant formula for the shearing strength is:

$$S_{\text{vane}} = \frac{2\ TM}{D^2\left(H + \frac{D}{4}\right)}$$

Where,

TM = Maximum torque
D = Diameter or overall width of the vane
H = Height of the vane.

The unconfined compressive shear strength was computed using the conventional expression:

$$S_{uc} = \frac{q_u}{2}$$

where q_u is the unconfined compressive strength, which is defined as the maximum applied load divided by the average cross section of the specimen.

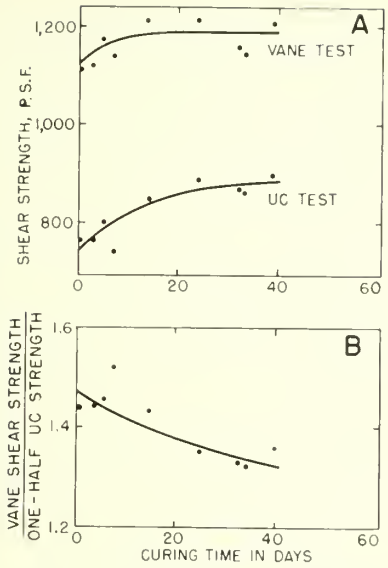

Figure 3.—Effect of curing time on shear strength of remolded clay loam specimens, test 2e.

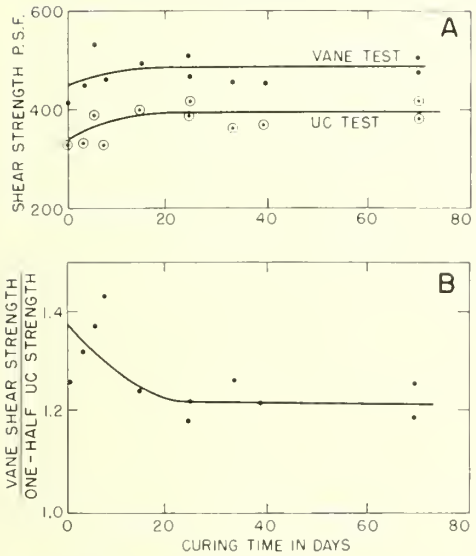

Figure 4.—Effect of curing time on shear strength of remolded clay specimens, test 9b.

Soil Preparation and Initial Testing

The 10 soils tested were four clays, two silty clay loams, a clay loam, a loam, and two sandy loams. The types of tests performed, the test results, and the mechanical properties of the soils are summarized in table 1. The soil to be used for the remolded tests was air dried, pulverized to break up clay lumps, and generally sieved through the No. 10 sieve to remove coarse particles—one exception, a silty clay loam, was sieved through the No. 4 sieve. Water was then added and thoroughly mixed with the soil to bring it to the selected moisture content. The mixture was then stored in a moist cabinet for at least 24 hours before it was molded and tested.

The remolded specimens were prepared in a mold that was 2 inches in diameter and 5 inches long. The soil was put in the mold in small increments and manually tamped with a wood rod three-fourths of an inch in diameter. The density of duplicate specimens could be reproduced within a range of 1.0 p.c.f. with this method. The vane shear tests generally were performed on these specimens while each was still in the mold. The vane was inserted into the soil until the top of the vane was approximately one-half inch below the top of the specimen. After the vane test, the specimens were pushed from the mold and trimmed to a length of 4 inches for the unconfined compression test. The tests listed under Group A in table 1 were made by using this procedure. For each soil molded into test specimens at moisture contents approximately between the plastic and liquid limits, the ratio of the vane shear strength to one-half the unconfined compressive strength (represented by vane-*uc* ratio) was constant; this ratio, however, varied from soil to soil and ranged from 0.6:1 to 1.4:1. These vane-*uc* ratios also are given in table 1. Typical test results for one soil, which show this constant ratio, are plotted in figure 2. Duplicate tests were made on this soil to evaluate different vane designs and the result was a seemingly excessive replication at each of the three moisture contents at which specimens were molded.

Four possible causes for the different shear strengths measured by the vane and unconfined compression tests are as follows: (1) Nonuniformity of soil structure; that is, particle arrangement and moisture distribution within the specimen; (2) variations in pore-pressure developments during shear; (3) progressive failure in the vane test; and (4) effects of testing procedures. Additional tests were made to investigate these factors.

Structure

Any nonuniformity of structure or strength within the test specimen would be reflected in the ratio of the vane strength to the unconfined compressive strength. This vane-*uc* ratio would be greater than 1:1 because the vane shears the soil specimen along a fixed surface, whereas the unconfined compression test allows the soil specimen to shear along the weakest surface. Shearing along a fixed surface tends to produce an average shear strength; however, shearing through the weakest part of a specimen produces the minimum strength.

It was hypothesized that a remolded test specimen has a less uniform structure than an undisturbed test specimen. If this were so, the vane-*uc* ratio for the specimens of remolded soil would be greater than the vane-*uc* ratio for the specimens of undisturbed soil. This hypothesis was investigated. Samples of an undisturbed silty clay (test 10a of Group B, table 1) were used; they were obtained from a 12-foot excavation with 3-inch and 6-inch sampling tubes. Unconfined compression test specimens 2 inches in diameter by 4 inches long were trimmed from the samples and tested. The vane-*uc* ratio for this undisturbed soil was about 0.90:1. This material was then thoroughly remolded and tested (test 10b). The resultant vane-*uc* ratio was about 1.2:1. This larger vane-*uc* ratio for the remolded soil appears to support the hypothesis that these ratios for remolded soil samples are less uniform than for undisturbed samples.

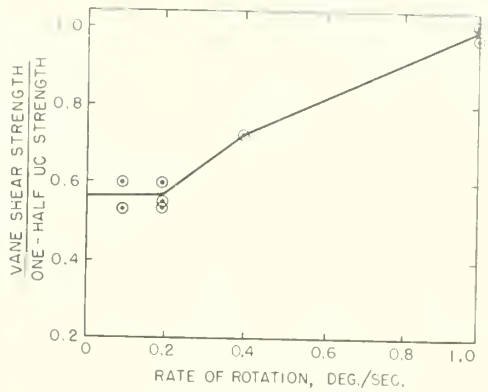

Figure 5.—Effect of rate of rotation on vane-uc ratio for sandy loam, test 8b.

As a further check of the effect of structure uniformity on the vane-*uc* ratio, the effect of age of the molded specimens was studied. It was hypothesized that soil of remolded samples will become more uniform as they are aged and, therefore, the unconfined compressive strength will increase and consequently the vane-*uc* ratio will decrease. This hypothesis was investigated for specimens of clay loam and clay, which were all molded at one time, immediately pushed out of the mold, wrapped in aluminum foil, and stored in a moist cabinet until they were tested.

Data shown in parts A of figures 3 and 4 show strength as a function of curing time and parts B of the same figures show the corresponding changes in the vane-*uc* ratio. A tendency can be seen for both the vane and the unconfined compressive strengths to increase as curing time is increased. However, the unconfined compressive strength increases at a more rapid rate, which causes a decreasing vane-*uc* ratio. This tendency was more pronounced for the clay loam than for the clay. The vane-*uc* ratio decreased from about 1.47:1 at zero time to about 1.33:1 at 39 days for the clay loam (test 2e) and from about 1.37:1 at zero time to about 1.22:1 at 39 days for the clay (test 9b). Based on these results, it appears that a remolded soil becomes more uniform with age; therefore, the difference between shear strengths measured by the two tests was reduced.

Pore Pressure

The second factor investigated was the possibility of pore-pressure buildup during shearing. This was investigated indirectly by studying the effect of rate of shear. A nearly constant rate of shear for the vane test was maintained by rotating the crank handle approximately one revolution per minute, thus obtaining an almost constant rate of stress increase and very little strain as the load was increased. As the load approached the shear strength of the soil, the result was a constant rate of strain of approximately 0.2 degree per second.

The effect of shear rate was investigated by varying the rate of shear at failure between 0.1 and 1.0 degree per second. These effects are recorded in Group C of table 1. A clay loam (test 2f) showed no measurable differences in strength for this range of shear rates.

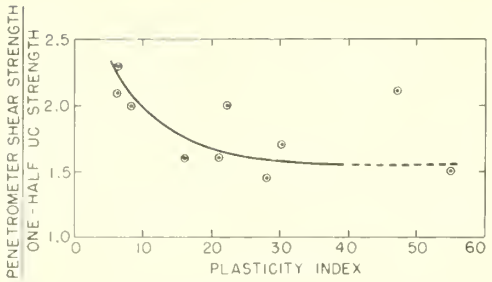

Figure 6.—Penetrometer-uc ratio versus plasticity index.

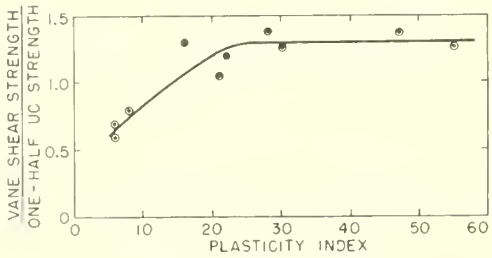

Figure 7.—Vane-uc ratio versus plasticity index.

However, data from test 8b recorded in figure 5 shows that a loam had a vane-*uc* ratio of 0.6:1 at 0.2 degree per second and of 1.0:1 at 1 degree per second. At rates slower than 0.2 degree per second, the vane-*uc* ratio was constant at a value of slightly less than 0.6:1. This apparent constant ratio could have been the result of the difficulty in manually rotating the crank at a constant rate at these slow speeds and the resultant variability in readings. Because 0.2-degree-per-second rotation gave the minimum strength, this rate was considered satisfactory for all tests.

As the loam is a dilatant soil, it is possible that negative pore stresses were induced during shear deformation. The negative pore-water stresses would create or increase normal pressure on the ruptured surface. This would, in turn, possibly increase an intergranular friction force causing an increased shear strength with increased rate of shear.

Progressive Failure

The third factor, the possibility of progressive failure in the vane test, was also studied indirectly. If a progressive failure were taking place in the vane test, a six-bladed vane would create a larger shear surface and cause a higher shear strength than a two- or four-bladed vane. The vane that was supplied with the vane equipment had four blades, each one-half inch high and one-fourth inch wide. Vanes having two and six blades were constructed for use in the tests reported here.

Tests made with the three different vanes produced a very small percentage difference in the measured strength of the soils—not enough to warrant any change from the standard four-bladed vane. The average strengths measured with the two-bladed vane were least; the measurements obtained with the standard four-bladed vane were 2 percent greater and those obtained with the six-bladed vane were 9 percent greater. But, in soft silty soils, the six-bladed vane caused significant compression of the soil during its insertion. The effect of this disturbance on the measured strength is unknown.

The effect of length of blades was also studied. Vanes having blades 1 and 2 inches long were built and the strengths obtained with them were compared with those obtained with the original vane having the ½-inch blades. The calculated shear strengths were essentially equal. As the tests made with the six-bladed vane produced shear strengths slightly greater than the four- and two-bladed vanes, some progressive failure appeared to be taking place.

Effect of Testing Procedure

The fourth factor that possibly affects the shear strength, as measured by the vane and the unconfined compression test, is the testing procedure used. As both the vane and the unconfined compression tests were often run on the same test specimen, there was a possibility that disturbance by the insertion of and testing with the vane would reduce the unconfined compressive strength. To investigate this, duplicate samples were periodically tested by only the unconfined compression test. Strengths obtained by the two methods checked very closely and, based on

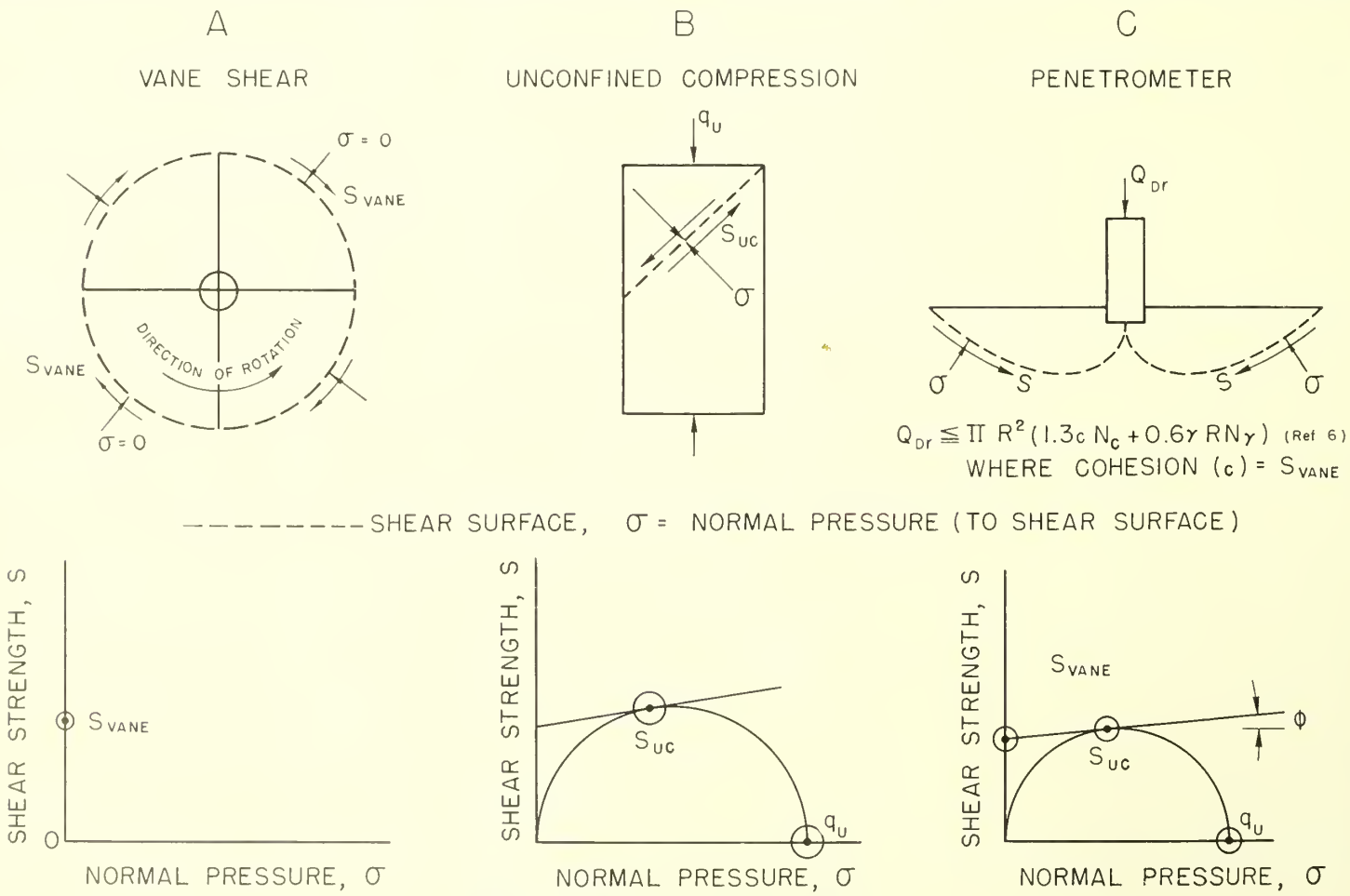

Figure 8.—Theoretical modes of failure.

Table 2.—Measured and computed force to insert penetrometer

Soil test	Cohesion, c	Angle of intergranular friction, ϕ	Measured force required to insert penetrometer	Computed force from Terzaghi's formula
No.	*P.s.f.*	*Degrees*	*Pounds*	*Pounds*
3a	300	23.0	3.50	3.00
3b	77	13.5	.62	.43
8a	192	26.5	2.27	2.24

these results, it was concluded that the insertion of the vane made an insignificant difference.

To determine whether any changes in strength were the result of removing the soil from the mold, tests were made with the vane on duplicate samples in the mold and after their removal from the mold. The duplicate samples were removed from the molds, wrapped in aluminum foil, and held by hand, while the vane test was being made in accordance with normal procedures. No measurable difference in shear strengths was observed for the soils, whether tested in or out of the mold, at moisture contents between the plastic and liquid limits.

Although the vane-*uc* ratio was constant for each soil within its plastic range, it increased as the moisture content decreased below the plastic limit. This may have been caused by crumbling of the soil and subsequent reduction of the unconfined compressive strength. When the vane test was conducted on these drier specimens after their removal from the mold, the specimens tended to crack during the test and the result was a reduced vane shear strength. The vane-*uc* ratio then approached the constant ratio obtained in tests for the soils having higher moisture contents.

Penetrometer Tests

In conjunction with the laboratory vane test evaluation, shear strengths were also obtained by using a commercially manufactured penetrometer. This device is used to measure the resistance of a soil mass to the penetration of a rod 0.245 of an inch in diameter, at ¼-inch penetration. Penetrometer tests usually were made with each vane shear test. The ratios of the average shear strengths obtained from the penetrometer to the strengths obtained from the unconfined compression tests are given in table 1. This ratio ranged from 1.3:1 to 2.3:1. The relationship between this ratio and plasticity index is shown in figure 6.

Mode of Failure

The wide variation in the vane-*uc* ratio, from 0.6:1 to 1.4:1, indicates that there is a basic difference in the mode of failure between the two tests. As shown in figure 7, the vane-*uc* ratio is related to the plasticity index. As the plasticity index increased, the vane-*uc* ratio increased to about 1.4:1. For a plasticity index of 14, the ratio was 1.0:1.

As noted previously, test specimens having nonuniform structure, water content, and density would tend to have vane-*uc* ratios greater than 1.0:1. It is likely that as the plasticity index of the soil increased, the molded test specimen became less uniform. For the highly plastic soils, the average strength measured by the vane test exceeded the strength measured by the unconfined compression test by 40 percent.

The vane-*uc* ratios of less than 1.0:1, indicating greater shear strengths obtained by the unconfined compression test, may have been caused by the inclusion of an intergranular friction component mobilized in the unconfined compression test but not mobilized in the vane test. Parts A and B of figure 8 represent the theoretical mode of failure for the vane and the unconfined compression tests, respectively. In the vane test, if the normal pressure (σ) induced during shear were zero, the shear strength would be a function of cohesion only and would not be affected by intergranular friction. In the unconfined compression test, the shear strength was a function of both cohesion and friction. It was reasonable therefore to expect that as the plasticity index of the soil became smaller, the frictional component would increase in significance and the result would be smaller vane-*uc* strength ratios.

If the assumption were true that the vane measures cohesion only, a useful Mohr diagram could be plotted using results of the vane and the unconfined compression tests. By using the vane measured strength at a zero normal stress and the Mohr circle determined by the unconfined compression test, the Mohr Envelope was established, as shown in figure 9. To check the validity of these Envelopes a comparison was made of the resistance to penetration obtained by use of the penetrometer with the resistance computed by using the Envelopes. The penetrometer resistance was calculated by Terzaghi's formula (*6*),

$$Q_{Dr} \leqq \pi R^2 \, (1.3cN_c + 0.6\gamma R N\gamma)$$

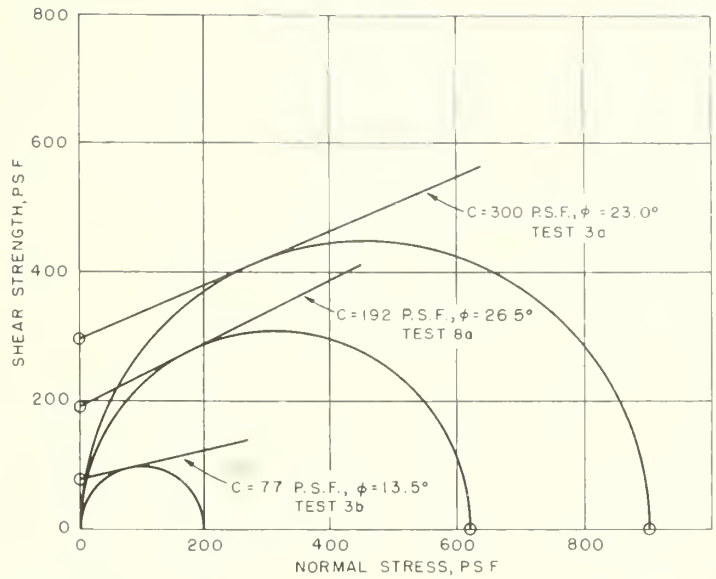

Figure 9.—Interpretation of shear strength measured in vane shear and unconfined compression tests.

where,

Q_{Dr}, critical load on a circular footing, lb.;
R, radius of footing, ft.;
c, cohesion, p.s.f. taken from vane shear strength;
N_c, $N\gamma$, bearing capacity factors dependent upon angle of internal friction, ϕ from unconfined compression test;
γ, unit weight of soil, p.c.f.

The theoretical mode of failure for the penetrometer is shown in part C of figure 8. A comparison of the computed penetration resistance with the measured forces is given in table 2. The reasonably good agreement indicated that results obtained by use of the Mohr Envelopes might be correct.

REFERENCES

(*1*) *Field Vane Shear Tests of Sensitive Cohesive Soils*, by Hamilton Gray, American Society of Civil Engineers Proceedings, vol. 81, No. 755, July 1955.

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(*3*) *The Vane Borer: An Apparatus for Determining the Shear Strength of Clay Soils Directly in the Ground*, by Lyman Cadling and Sten Odenstad, Royal Swedish Geotechnical Institute Proceedings No. 2, Stockholm, 1950.

(*4*) *Consistency Limits of Clay by the Vane Method*, by M. Darienzo and E. Vey, Proceedings Highway Research Board, vol. 34, 1955, pp. 559–566.

(*5*) *Vane Tests in the Alluvial Plain of the River Forth Near Grangemouth*, by A. W. Skempton, Geotechnique, vol. 1, No. 2, December 1948, pp. 111–124.

(*6*) *Theoretical Soil Mechanics*, by Karl Terzaghi, John Wiley & Sons, New York, N.Y., 1943, p. 125.

Evaluation of Results of Laboratory Vane Shear Tests With Pore-Pressure Measurements

by [1] **Professor NYAL E. WILSON, McMaster University, Hamilton, Ontario, Canada**

The following comments on the preceding article, *Evaluation of Laboratory Vane Shear Test*, are those made by Professor Wilson.

"Gonghmour and Sallberg proposed two hypotheses regarding the research work on the laboratory vane-shear test; these hypotheses are concerned with (1) the influence of pore-water pressures and (2) the effects of progressive failure. Some interesting research work has been conducted on the laboratory vane shear test and the results of this work substantiate the findings of Gonghmour and Sallberg.

"This research involved using the laboratory vane shear apparatus in dilatant soil; the soil used was a medium-fine silt. The rate of testing was accurately controlled by a variable-speed motor and the vane blade was instrumented so that pore-water pressures could be measured on the shear surface (fig. 1). The pore-pressure measurements were taken by welding hypodermic tubing to the edge of one of the vane blades; the end of the tubing was slotted and covered by a No. 200 mesh screen.

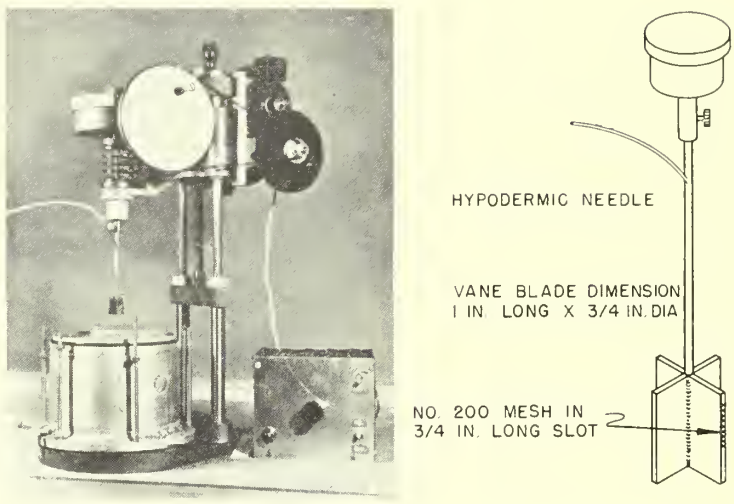


Figure 1.—Laboratory vane apparatus and variable speed motor.

Influence of Pore-Water Pressures

"As in the research by Gonghmour and Sallberg, it was found that the torque applied to the vane shaft was dependent upon the speed of testing. Figure 2, showing torque versus testing speed for vane tests in silt, indicated that a higher torque was associated with higher testing speeds.

"The value of the torque was overestimated by about 25 percent when tested at the usual speed in the laboratory; this overestimation was related to the particular torsion spring used in the test. The deformation of the soil was dependent upon the speed of testing; that is, the angular velocity of the torque dial, and it was related to the rigidity of the torsion spring and varied with each apparatus; this is one of the disadvantages with the laboratory vane test, which is neither rigorously stress-controlled nor strain-controlled.

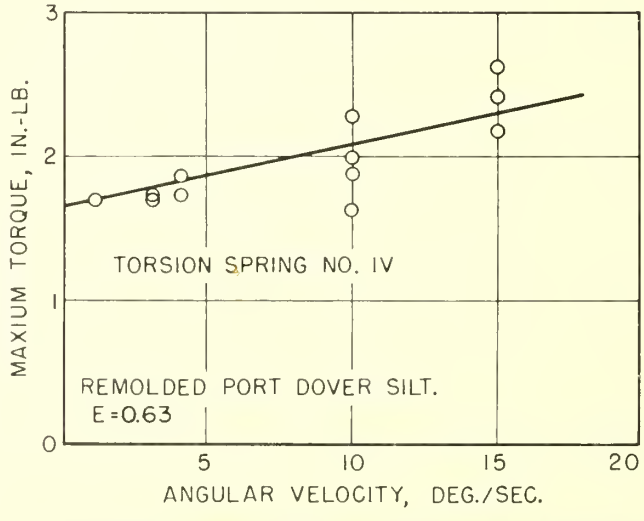


Figure 2.—Dependence of maximum torque on angular velocity of torque dial in vane tests on silt.

"To investigate the influence of pore-water pressure on the torque applied to the vane shaft, a series of tests was conducted in silt (fig. 3); these tests, which were conducted at constant speed, also indicated that a change in testing depth from 2 inches to 3 inches had no significance. The results showed that induced pore-water pressures can be in either the negative or positive range depending on the formation of the menisens at the start of the test. Negative pore-water pressures were applied to the soil to determine the influence over a greater range. The sloped line (fig. 3) indicated that the laboratory vane test, commonly considered as an "undrained" test, acquired the characteristics of a "drained" test in dilatant soils; this anomaly has also been found for tri-axial tests on dilatant soils.

Progressive Failure

"Observations were taken during the tests to investigate progressive failure. The vane was inserted to the depth of the vane blades and photographed as the torque was applied and as the angular deformation took place (fig. 4). At a strain of 10° and when the maximum torque occurred, shear surfaces were generated at the tips of the blades and at right angles to them. At this stage in the

[1] Presented at 43d annual meeting, Highway Research Board, Washington, D.C., Jan. 1964

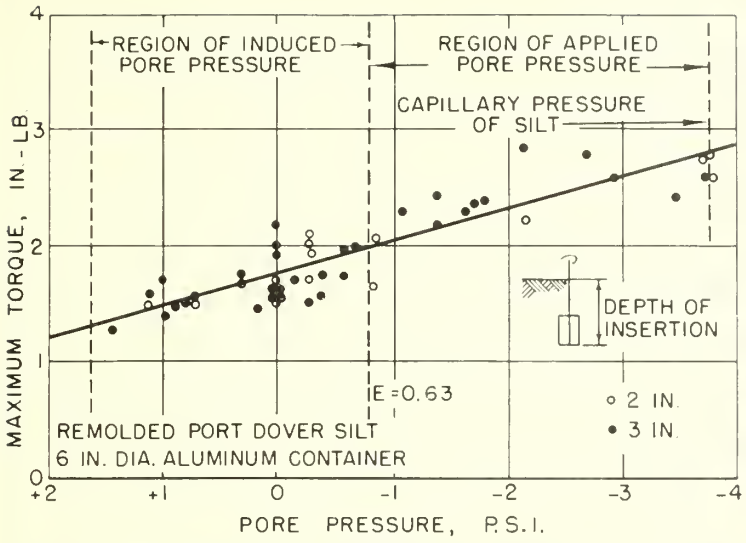
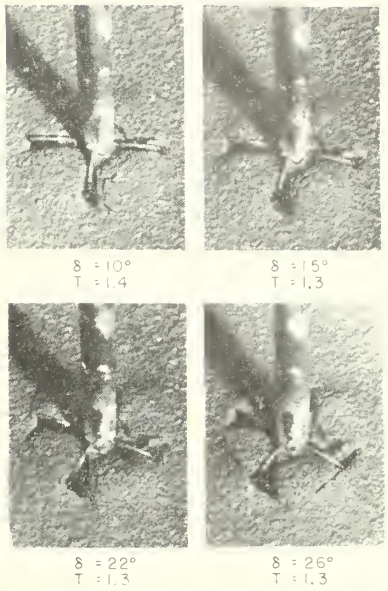


Figure 3.—Maximum torque versus pore pressure from vane tests on silt.



δ = ANGULAR DEFORMATION
T = TORQUE, IN.-LB.
REMOLDED PORT DOVER SILT

Figure 4.—Vane test in silt.

test, the shear surface was not cylindrical but almost rectilinear. As the angular strain increased and the torque decreased, the shear surfaces extended until, ultimately, a cylindrical failure surface was formed. During these stages of the test, voids were formed behind the vane blades and the shearing resistance was zero at these voids. Even though the ultimate shearing surface was cylindrical, it was not necessary for the shear surface at maximum torque to be cylindrical or the stress distribution on the walls of the cylinder to be uniform. As the stress distribution at the shearing surface and along the vane blades was unknown, it was not possible to use the vane with any accuracy in this type of soil."

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(2) *Techniques of Field Measurement and Sampling,* by N. E. Wilson, Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris, 1961.

(3) *The Vane Borer: An Apparatus for Determining the Shear Strength of Clay Soils Directly in the Ground,* by Lyman Cadling and Sten Odenstad, Royal Swedish Geotechnical Institute Proceedings No. 2, Stockholm, 1950.

(4) *Laboratory Vane Shear Tests and the Influence of Pore-Water Stresses,* by N. E. Wilson, ASTM Shear Testing Symposium, 1963.

Comments by Goughnour and Sallberg

Comments of Goughnour and Sallberg regarding Professor Wilson's evaluation were, as follows:

"The authors are grateful to Professor Wilson for his interesting and informative discussion. His development of the pore-pressure device is a welcome contribution. It would be of interest to continue the study of pore-pressure on a variety of soils having a wide range of plasticity indexes.

"Professor Wilson refers to the 'usual speed' of vane testing, but in figure 2 shows values up to 15 degrees per second angular velocity of torque dial. These values appear to be much higher than values of normal testing.

"In his last sentence, Professor Wilson states, 'As the stress distribution . . . was unknown, it was not possible to use the vane with any accuracy in this type of soil.' It should be pointed out that the stress distribution is not known for any type of test. It would seem that the vane test could be considered as accurate as any other of the commonly used tests."

NEW PUBLICATIONS

Opportunities for Young Engineers

A revised and updated version of *Opportunities for Young Engineers in the Bureau of Public Roads (1964)* has recently been issued by the Bureau of Public Roads, U.S. Department of Commerce. This 16-page, illustrated pamphlet may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for 15 cents a copy. The introduction to this pamphlet tells of the Bureau's responsibilities and its cooperation with other Government agencies and foreign countries.

This publication is intended to provide information for college students interested in careers in highway or highway-bridge engineering. Information is presented on the 3-year training program for young engineers and on the organization, operations, and history of the Bureau.

Additional information regarding opportunities in the Bureau of Public Roads is contained in three folders that may be obtained free of charge by writing to the Personnel and Training Division, Bureau of Public Roads, Washington, D.C., 20235. These publications are: (1) *Opportunities as a Professional Highway Engineer*, (2) *Professional Challenge and Growth for Accounting Majors*, and (3) *Challenging Opportunities in the Bureau of Public Roads Right-of-Way Training Program.*

Typical Pedestrian Bridges

The fifth volume in the Bureau of Public Roads series on *Standard Plans for Highway Bridges (1962)*, Vol. V—*Typical Pedestrian Bridges*, is expected to be issued late in December or early in January 1965 and will be for sale by the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402.

The plans for pedestrian bridges are intended for the guidance of State, county, and local highway departments in the development of suitable and economical highway bridges across the Interstate or dual-highway systems. The plans presented in this publication and the first four volumes should be particularly useful to the smaller highway departments having limited engineering staffs. An effort has been made to provide sufficiently complete information on all bridge plans so that they will approach contract drawings as nearly as practicable. For any given location, however, the requirements imposed by the site conditions probably will necessitate modification of the bridge plans because they have been developed generally only for right-angle crossings, level roadway grades, and typical walkway vertical curves. However, to improve the educational value of volume V, calculations for several of the more difficult to design pedestrian bridges have been included in this publicaion. The inclusion of these calculations will serve as examples and encourage the use of the most suitable bridge for the location for which it is intended. Typical examples of pedestrian bridges are presented. A highway having two, 3-lane roadways separated by a 40-foot median was considered to be the average roadway requiring a pedestrian crossing, and these bridge plans have been presented so that they can be adapted for many situations. All examples shown have been selected on the basis of esthetics and economic suitability.

The inclusion of two-span continuous bridges has been the direct result of a recent study that showed a favorable economical comparison between two-span and four-span continuous bridges. Removing the shoulder pier creates an impression of openness, which greatly enhances the esthetic qualities of the bridge. In addition, it increases sight distance, increases safety, and improves conditions for maintenance. It is assumed that normally the pedestrian bridge can be placed in a cut section or ramp on earth embankment. For those bridges that require stairs or ramps in extremely limited space, stairway and ramp alternates have been detailed and can be adopted for most all bridges.

Appendix A presents a rather complete design criteria for use in the design of pedestrian bridges. It is believed that the design requirements for these bridges should be more liberal than for highway bridges. Therefore, the design criteria reflects a considerable number of exceptions to the AASHO specifications.

Dimensions and Weights of Vehicles

A report prepared by the Bureau of Public Roads, *Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems*, has been transmitted to the Congress by the Secretary of Commerce and has been published as House Document No. 354 (1964). Copies of H. Doc. No. 354 are sold by the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402, for 45 cents each.

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PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–32 are available upon request addressed to Bureau of Public Roads, Washington, D.C., 20235.*

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

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1951, 35 cents. 1955, 25 cents. 1958, 30 cents. 1959, 40 cents. 1960, 35 cents. 1962, 35 cents. 1963, 35 cents. (Other years are now out of print.)

REPORTS TO CONGRESS

Factual Discussion of Motortruck Operation, Regulation and Taxation (1951). 30 cents.

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:

First Progress Report, House Document No. 106 (1957). 35 cents.

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways. 1937–61. $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

Calibrating and Testing a Gravity Model With a Small Computer (1964). $2.50.

Catalog of Bridge Plans. $1.00.

Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

Federal Laws, Regulations, and Other Material Relating to Highways (1960). $1.00.

Financing of Highways by Counties and Local Rural Governments: 1942–51 (1955). 75 cents.

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PUBLICATIONS—Continued

Highway Finance 1921–1962 (a statistical review by the Office of Planning, Highway Statistics Division). 15 cents.

Highway Planning Map Manual. $1.00.

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

Highway Research and Development Studies (1964). $1.00.

Highway Statistics (published annually since 1945):
1956, $1.00. 1957, $1.25. 1958, $1.00. 1959, $1.00. 1960, $1.25. 1961, $1.00. 1962, $1.00.

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Peak Rates of Runoff From Small Watersheds (1961). 30 cents.

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Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1958: a reference guide outline. 75 cents.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects. FP–61 (1961). $2.25.

Standard Plans for Highway Bridges (1962):

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Vol. IV—Typical Continuous Bridges. $1.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Aerial Surveys in Highway Engineering (1960). 40 cents.

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Traffic Safety Devices, Directory of National Organizations (1963). 15 cents.

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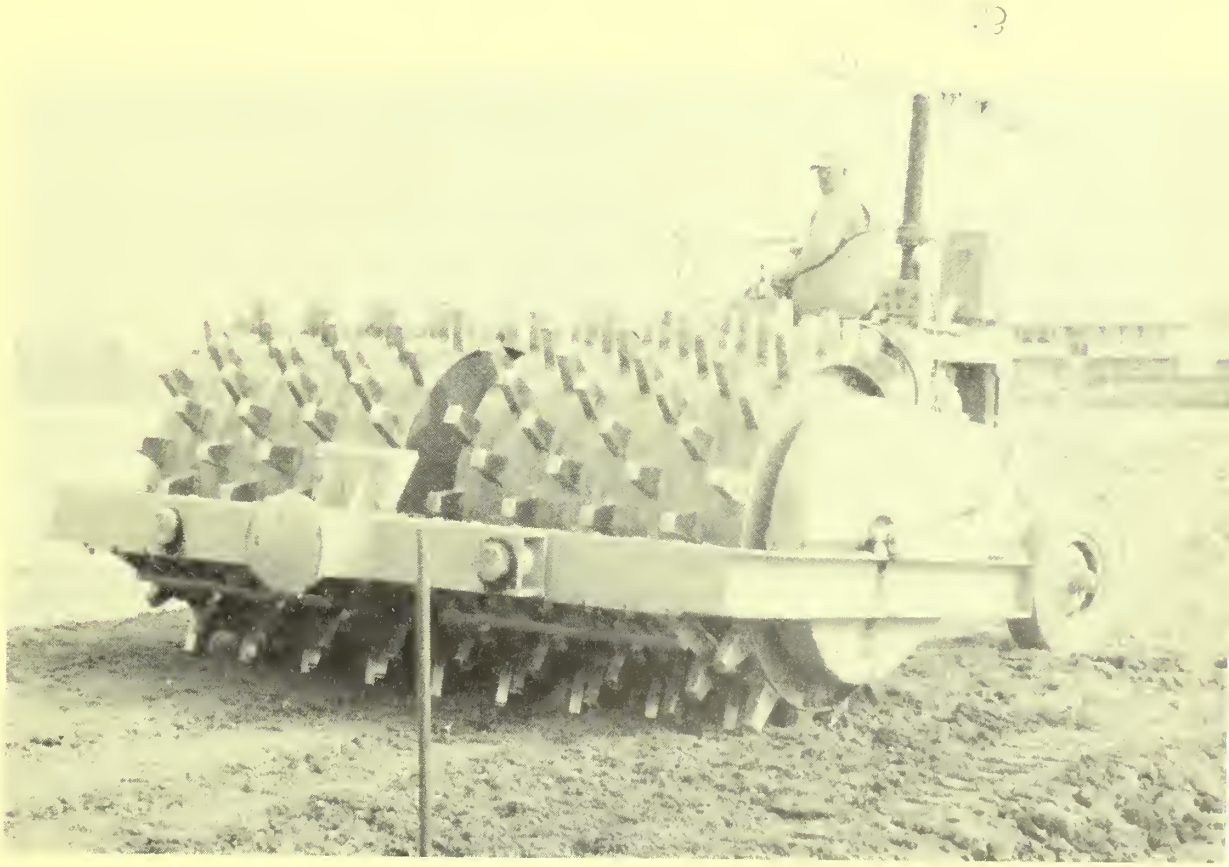

Compaction of a 6-inch silty-gravel test course by a dual-drum self-propelled sheepsfoot roller. Compaction is being done as part of a pilot study for an HPR cooperative compaction research project at Hazelcrest, Illinois.

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IN THIS ISSUE

Shrink-Swell Potential of Soils, *by C. W. Ring III* 97

Friction Reducing Mediums for Rigid Pavement Subbases, *by A. C. Timms* 106

Simplified Procedures for Determining Travel Patterns Evaluated for a Small Urban Area, *by C. Ben, R. J. Bouchard, and C. E. Sweet* 112

New Publications 121

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 6, 1961.

Content of this publication may be reprinted. Mention of source is requested.

Shrink-Swell Potential of Soils

BY THE STRUCTURES AND APPLIED MECHANICS DIVISION BUREAU OF PUBLIC ROADS

Reported by [1] GEORGE W. RING, III, Highway Research Engineer

This article describes a laboratory study conducted to develop a new test method for measuring the shrink-swell potential of a soil, independent of its molding moisture and density. The method consists of measuring the volume change that occurred when the soil was dried after cyclic wettings and dryings to achieve an equilibrium condition. As the shrink-swell potential test requires from 1 to 100 weeks to complete, swell-potential test results on 12 soils were compared to results obtained by 8 standard test methods in an attempt to find a rapid and reliable substitute. Good relationships were obtained with plasticity index, Georgia volume change, surface area, and linear shrinkage. Of these eight, linear shrinkage shows the most promise as a rapid, reliable substitute for the longer shrink-swell potential method.

Introduction

THE PERFORMANCE of engineering structures is sometimes affected by the shrinking and swelling of soils. Pavements become wavy as the result of differential volume change of the subgrade. Uneven shrinking or swelling of the foundation soils cause unsightly and dangerous cracks to develop in buildings. Pipelines are deformed, misalined, and occasionally ruptured by volume change of the soil in which they are embedded. Sometimes concrete canal linings crack when water seeps through construction joints and causes uneven swelling of the underlying clays that have a high change of volume. Engineers need to know whether the soils to be used in construction may create problems because of excessive volume change.

This is a report on a laboratory study to develop a method of evaluating the shrink-swell potential of a soil—a measure of how much the soil may change in volume as moisture content changes. The report includes (1) a brief literature survey reviewing mechanics of volume change of soils, (2) results of a new test developed to measure volume change of soils when they are alternately wetted and dried (shrink-swell potential), and (3) a comparison of the results of this test to many standard tests used to measure shrinking and swelling characteristics of soils.

When a soil swells or shrinks, three primary actions appear to occur. These, as illustrated in figure 1, are: (1) elastic bending or unbending of soil particles, (2) interlayer expansion or contraction of certain layered clay minerals, (3) osmotic imbibition, which is a change in the thickness of water films on the exterior surfaces of soil particles. All of these are usually associated with changes in moisture content of the soil, although elastic bending may occur to a limited extent without moisture change. There may or may not be an associated change in the volume of the air in the soil.

Mechanisms of swelling have been studied by many investigators. Gilboy (*1*) [2] suggested that the amount of elastic deformation that occurs in a soil under a given load may depend on particle shapes that promote bending and unbending (figure 1, part A). His experiments with mixtures of mica and dune sand showed that the consolidation and rebound of the compacted mixtures were proportional to the amount of mica present. The results of his tests on three different mixtures are, as follows:

Mica in Mixture	*Decrease in Void Ratio Under 10 kg./cm.²*	*Increase in Void Ratio When Load is Removed*
Percent	*Percent*	*Percent*
10	36	26
20	47	31
40	51	42

From X-ray diffraction studies, Bradley, Grim, and Clark (*2*) noted that one clay mineral, montmorillonite, expanded by taking on molecular layers of water internally (fig. 1, part B). Fink and Thomas (*3*) studied this phenomenon and found that the unit thickness of one layer of a lithium bentonite (montmorillonite) crystal increased about 780 percent when allowed to absorb water. Ladd (*4*) showed that a saturated clay soil swelled less in salt solutions than in water. He attributed this to the fact that less liquid was imbibed (fig. 1, part C) when the soil was soaked in the salt solution. Compacted Vicksburg buckshot clay, soaked in a 5-molar solution of $CaCl_2$, swelled about 6 percent less than when it was soaked in water. Allen and Johnson (*5*) determined that the amount of swell of a compacted soil was greatly affected by the initial moisture condition of the soil and to a lesser extent by the initial density. A soil compacted in a dry condition swelled much more than when compacted to the same density in a wet condition. Similar results were obtained by Holtz and Gibbs (*6*).

Conclusions

On the basis of the new work reported herein, these conclusions have been made:

The objective of this study appears to have been reasonably well achieved: An alternate wetting-and-drying procedure was developed to establish the shrink-swell potential of soils. The wide range of soil types used for this study provided a broad enough base to indicate that the linear-shrinkage test has real potential as a substitute for the very time-consuming wetting-and-drying method.

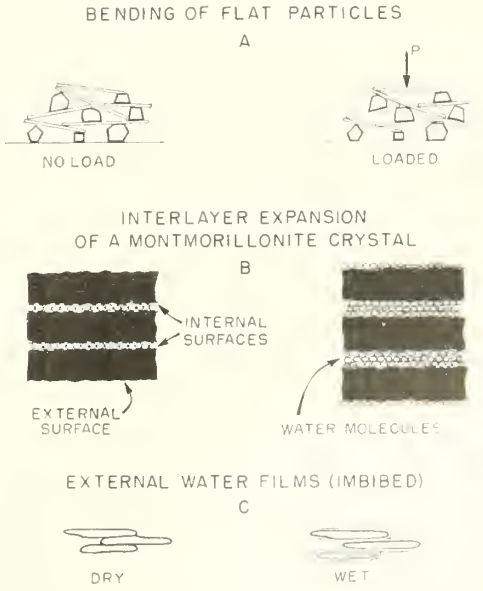

Figure 1.—Some mechanisms of volume change.

[1] Presented at the 44th annual meeting of the Highway Research Board, Washington, D.C., January 1965.

[2] References indicated by italic numbers in parentheses are listed on page 105.

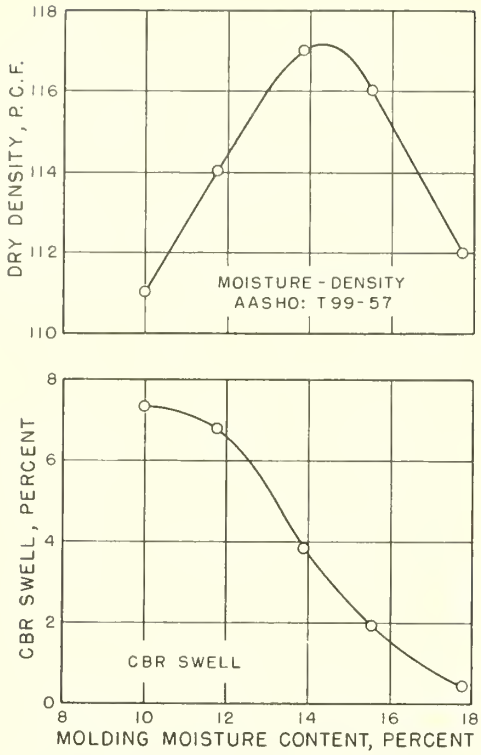

Figure 2.—Variation of swell with molding moisture and density of Hybla Valley clay.

Shrink-Swell Potential

The study discussed here is concerned primarily with the problem of predicting potential shrinkage and swelling of soils, as this activity affects roadways and other types of structures. In the bulk volume of a soil-water-air mixture, shrinkage is the decrease caused by drying, and swelling is the increase caused by wetting. Measuring an inherent property of soils, such as shrink-swell potential, is difficult, not only because of the many types of mechanical actions taking place but also because the measurements may be radically affected by environmental conditions. For example, figure 2 shows how the swelling of one soil, as measured during the California Bearing Ratio (CBR) test, changed as the compacted density and molding moisture content changed. It was necessary to minimize these effects to determine a true measure of shrink-swell potential.

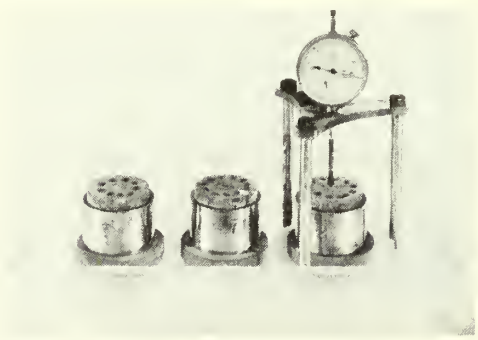

Figure 3.—Apparatus for cyclic wetting and drying test.

Existing laboratory test methods for determining the shrink-swell properties of soils usually fall into one of two categories: either a volume change test or a swell-pressure test. Information on 10 volume change tests and 4 swell pressure tests is shown in tables 1 and 2, respectively. Some volume change tests measure swelling; other volume change tests measure shrinkage. The Georgia method (*7*) for determining volume change measures both swelling and shrinkage. Two identical specimens are compacted, then one is soaked while the other is dried. The test result is determined from the combined volume change of the two specimens. H. C. Porter (*8*) suggests that alternately wetting and drying a soil rapidly may achieve an equilibrium condition, regardless of the initial moisture and density of the soil specimen.

Consequently, to determine the most suitable test for measuring shrink-swell potential, the Georgia volume change test and Porter's findings were investigated further. The tests were intended to be rapid and

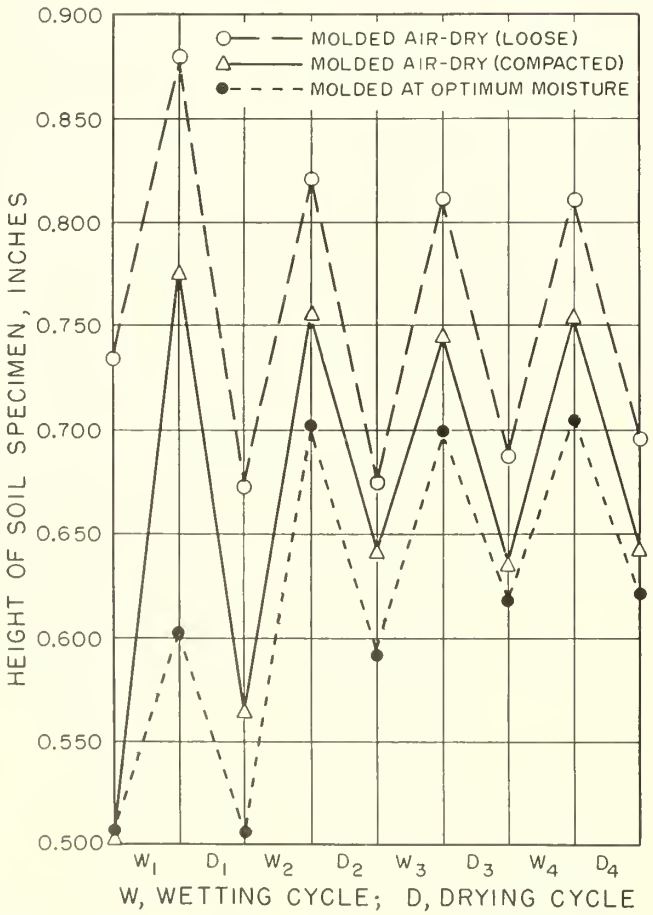

Figure 4.—Height change of 3 specimens of Iredell clay when alternately wetted and dried under a 0.25 p.s.i. surcharge.

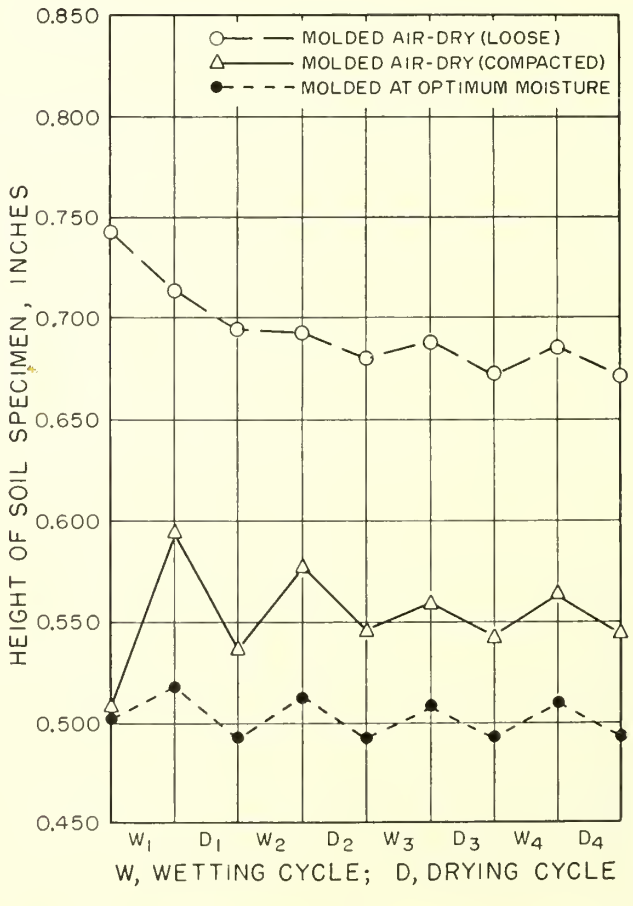

Figure 5.—Height change of 3 specimens of Cecil soil when alternately wetted and dried under a 0.25 p.s.i. surcharge.

Table 1.—Types of volume change tests

General type	Specific examples	Size of test specimen	Initial moisture content of soil	Soaking or drying time	Use of test results	Test procedures
Free swell	Method proposed by Winterkorn and Baver.	0.5 gram	Dessicated over P_2O_5.	24 hours	Identify swelling soils	Soil lightly tamped into glass Jena tube and permitted to absorb liquid of known dielectric constant. Test repeated with liquid having different dielectric constant. Relative amounts of the two liquids absorbed indicates swelling activity of soil.
	Free swell in graduate of water.	Variable, generally 1 to 10 grams.	Air dry	24 hours	Determine relative swelling potential of soils.	Soil slowly added to 100 ml. graduate of water. After soaking, volume of soil-water mixture is measured on the graduate.
Swell or consolidation with surcharge (confined laterally).	CBR	6-in. dia. by 4⅝- to 7-in. length.	Optimum (AASHO: T 99) or field moisture content.	4 days	Identify swelling soils for flexible pavement design.	Soil is compacted into mold, usually by dynamic compaction (for undisturbed samples, mold is forced into soil), trimmed, and soaked under a surcharge. Change in height is determined.
	AASHO: T 116	4-in. dia. by 1.5625-in. length.	Variable—usually at optimum moisture content (AASHO: T 99).	Until swell is no more than 0.001 in. in 18 hours; maximum 7 days.	Measure relative amount of swell for different conditions of moisture and density.	Similar to above, smaller mold, usually statically compacted.
	Different types of odometers.	Variable, usually from 1.5- to 4-in. dia. and 0.5- to 1.5-in. length.	Variable, to fit problem.	Variable	Determine rate and amount of consolidation or swell.	Similar to two procedures noted above, except specimens are smaller and test conditions are more variable.
Swell and shrinkage.	Georgia volume change.	4-in. dia. by 1-in. length.	Optimum (AASHO: T 99).	48 hours	Classify embankment soil, subgrade, and base course materials for pavement design.	Two identical specimens dynamically compacted. One specimen soaked and permitted to swell; the other specimen is oven dried at 110° C. Total volume change is then calculated.[1]
Volumetric shrinkage.	AASHO: T 92	1.75-in. dia. by 0.50-in. length.	Liquid limit to 1.1 x liquid limit.	Air dry to change in color, then oven dry at 110° C. to constant weight (usually 4 hours or more).	Calculate: (1) shrinkage limit, (2) shrinkage ratio, (3) volumetric change, and (4) linear shrinkage.	Soil-water mixture is placed into dish of known volume and tapped firmly to release entrapped air. After drying, volume of dry soil pat is determined by measuring displacement.
Linear shrinkage	Texas Bar Linear Shrinkage Test.	5-in. length by ¾-in. square.	Slightly wetter than liquid limit.	Air dry to change in color, oven dry at 105° C.	Identify high volume-change soils.	Soil is mixed with sufficient water so that a groove in a half-inch thick pat just closes without jarring. Soil is placed in rectangular mold and dried. Length of dry soil is measured and compared to original length.
	Saturated linear shrinkage.	10-in. length by 1-in. dia. (semi-circular).	Sufficient to provide free water after thorough mixing.	Air dry at 70° F. for 4 hrs., then oven dry at 105° C. for 24 hours.	Identify high volume-change soils.	Soil is mixed with sufficient water until free water is visible on surface. Thirty minutes after mixing, excess water is decanted; soil is remixed and placed in mold. Dry length of soil is compared to original length.
Change in thickness of clay mineral crystal.	X-ray diffraction test.	A few milligrams.	Variable	Not applicable	Study primarily of elemental soil properties.	Measured reflections from a beam of X-rays directed at different angles onto thin layer of soil (clay) shows thickness of basal layer spacings in crystalline structure.

[1] Total volume change (V) is calculated:

$$V = \left(\frac{V_2 - V_1}{V_1} + \frac{V_3 - V_4}{V_3} \right) 100$$

where,
V_1 = Original volume soaked specimen.
V_2 = Final volume of soaked specimen.
V_3 = Original volume of dried specimen.
V_4 = Final volume of dried specimen.

simple. The first tests were made by the Georgia method. In these tests, some medium to low plasticity soils exhibited high swelling characteristics when soaked, evidently as the result of the compaction of the specimen. Also, some high plasticity soils swelled much more during a second soaking than during the first soaking period. Because of these occurrences, an approach based on Porter's studies was thought to possibly be more logical. A test procedure was devised to determine whether alternate wetting and drying, as suggested in Porter's studies, would achieve an equilibrium condition of shrinking and swelling, regardless of initial moisture and density condition. In this procedure, three specimens of each soil were molded to the initial moisture and density (percentage of AASHO: T 99) conditions, as follows:

Specimen	*Moisture content*	*Density percent*
1	Optimum	100
2	Air dry	100
3	Air dry	64-83

Specimens No. 1 and 2 were compacted statically to a height of 0.5 inch and a diameter of 2.0 inches. The height of the uncompacted specimen (No. 3) depended on the

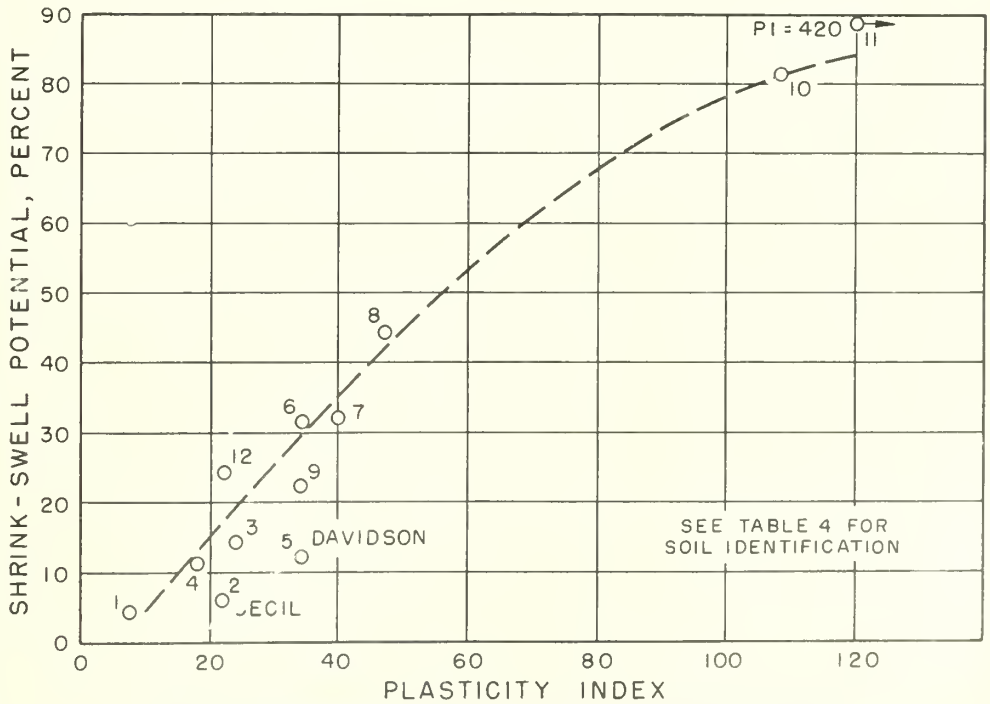

Figure 6.—Relation of plasticity index to shrink-swell potential.

Table 2.—Types of swell pressure tests

Test	Size of test specimen—diameter × length	Initial moisture content of soil	Soaking time	Change in height or expansion permitted—per 1 p.s.i. pressure	Use of test results	Comments on test
	Inches		*Hours*	*Inches*		
AASHO: T 190	4 × 2.5	Determined by exudation pressure criteria. Moisture content is usually close to optimum by AASHO: T 99.	16 to 20	0.00264	In thickness design of flexible pavements.	Specimen compacted by kneading compactor and static loading. Placed in expansion frame; swells in one direction against calibrated spring steel bar.
Strom and Hennes swell pressure test.	4 × 1.5 to 2.5	Variable	16	May be varied	Not stated	Compacted similar to above. Placed in pressure cell and surrounded by air or water, depending on amount of restraint desired. Side wall friction occurring in AASHO: T 190 claimed to be eliminated. Pressure measured by gage.
FHA swell index (soil PVC meter).	2.75 × 0.85	Air dry or at optimum (AASHO: T 99).	2	0.001	Identify high swelling or shrinking soils.	Specimen compacted into ring by impact compaction, trimmed to size. Soil expands against proving ring when soaked in water. The resultant swell index (pressure) is related to the potential volume change (noncritical; marginal; critical; very critical) of soil.
Odometer placed in universal testing machine.	Varied, but usually 1.5 to 4 × 0.5 to 1.5.	Varied to fit problem.	Variable [1]	0.0001 to 0.00001	Research, special problems.	Type of compaction, specimen size, soil density, and moisture conditions differ to fit equipment and/or problem.

[1] Usually no more than 1 week.

Table 4.—Shrink-swell potential of 12 soils

Soil		Shrink-swell potential
No.	*Type*	*Percent*
1	Portneuf	4.0
2	Cecil	5.7
3	Hybla Valley	13.9
4	Williams	10.8
5	Davidson	11.7
6	Parsons	13.3
7	Winterset	31.9
8	Iredell	46.9
9	Potomac	22.4
10	Ca Bentonite	81.3
11	Na Beutonite	89.0
12	Berthoud	24.0

soil type, as the soil for the specimen was poured loosely into the mold. The same amount of soil was used for all three specimens. After being molded, all the specimens were subjected to four cycles of alternate wetting and drying, figure 3. Wetting was continued on each soil until swelling apparently ceased. This period of time ranged from about 200 minutes for low plasticity soils to 200,000 minutes (140 days) for the bentonites. Drying was at 110° C. Although the drying temperature may be regarded by some as too severe, tests on Parsons and Davidson soils showed that this temperature had little or no effect on subsequent cycles of wetting and drying. The volume change

Table 3.—Test data for soils used in shrink-swell potential study

BPR sample No., soil, and source	Soil class		Mechanical analysis, soil finer than—(mm.)						Plasticity index	Liquid limit	Specific gravity	AASHO: T 99		Shrinkage		Linear shrinkage	Volume change		Shrink-swell potential	Surface area of material finer than 0.42 mm.		Clay minerals in material finer than 2 microns	
	AASHO	Unified	2.0	0.42	0.074	0.020	0.005	0.001				Optimum moisture content	Maximum dry density	Limit	Ratio		Georgia	AASHO: T 116		External	Internal	Predominant	Accessory
			Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*				*Pct.*	*P.c.f.*			*Pct.*	*Pct.*	*Pct.*	*Pct. of wet volume*	*m.²/g.*	*m.²/g.*		
S 30470, Portneuf silt loam, Idaho.	A-4(8)	ML-CL	100	99	97	51	22	13	4	26	2.72	16	107	19	1.73	3	4.3	7.0	4.0	25	27	Montmorillonite	Illite and kaolinite.
S 14335, Cecil clay, Ala.	A-7-5(17)	MH	100	91	65	50	38	28	22	63	2.84	27	90	25	1.57	12	14.6	3.8	5.7	35	18	Kaolinite	Degraded mica and free iron oxides.
S 35732, Hybla Valley clay, Va.	A-7-6(11)	CL	100	92	58	41	32	22	24	47	2.76	14	117	16	1.83	11	12.3	5.9	13.9	36	113	Montmorillonite	Small amounts of illite and kaolinite.
S 30038, Williams loam, N. Dak.	A-6(12)	CL	100	96	77	57	40	24	18	38	2.73	18	108	16	1.80	11	13.5	3.0	10.8	44	36	Montmorillonite	Illite and kaolinite.
S 37000, Davidson clay loam, Va.	A-7-5(20)	MH	100	99	95	88	80	74	34	70	2.89	30	91	24	1.60	14	12.3	1.5	11.7	48	39	Degraded mica	Kaolinite, amorphous iron oxides and quartz.
S 30375, Parsons silt loam, Okla.	A-7-6(20)	CH	100	99	94	75	59	53	34	62	2.65	28	91	9	1.98	19	29.2	6.0	31.3	47	106	Montmorillouite	Kaolinite.
S 30907, Winterset silty clay loam, Iowa.	A-7-5(20)	CH	100	99	98	80	52	42	40	70	2.73	25	94	9	2.02	21	25.2	11.0	31.9	51	141	Montmorillonite	Illite and kaolinite.
S 30932, Iredell silt loam, Va.	A-7-5(20)	MH-CH	100	97	91	80	71	60	47	82	2.85	25	94	9	2.07	26	46.1	15.1	46.9	80	157	Montmorillonite	Kaolinite and halloysite.
S 16313, Potomac clay, D.C.	A-7-6(20)	CH	100	100	97	87	52	41	34	60	2.83	22	99	16	1.88	19	22.9	25.3	22.4	33	93	Kaolinite	Illite and montmorillonite.
S 36004, Bentonite (calcium ion), Miss.	A-7-6(20)	CH	100	100	100	(1)	(1)	(1)	108	167	2.83	45	75	13	1.78	35	(2)	30.1	81.3	103	532	Montmorillonite	None.
S 36005, Bentonite (sodium ion), Wyo.	A-7-6(20)	CH	100	100	100	(1)	(1)	(1)	421	515	2.81	51	69	36	1.23	39	(2)	28.5	89.0	51	521	Montmorillonite	None.
S 35332 and S 35365 (combined), Berthoud, Mont.	A-7-6(14)	CL	100	100	95	79	64	30	22	44	2.74	20	102	13	1.96	16	22.0	13.3	24.0	42	105	Vermiculite and montmorillonite.	Kaolinite and illite.

[1] Could not be determined by hydrometer method. (2) Not determinable by Georgia method.

Table 5.—Relationships of standard test results to shrink-swell potential

Test method or measurement	Quality of relationship to shrink-swell potential	Remarks
Plasticity Index.	Good	May overestimate shrink-swell potential of soils containing iron oxides and inactive clays.
Shrinkage limit.	Fair	Underestimates shrink-swell potential of bentonitic soils.
AASHO: T 190.	Poor	Molding moisture and density conditions not suitable for prediction of shrink-swell potential.
CBR	Fair	Relationship with shrink-swell potential slightly improved for specimens molded to AASHO: T-180.
AASHO: T-116.	Fair	May overestimate the shrink-swell potential of soils sensitive to method of compaction.
Georgia volume change test.	Good	May overestimate shrink-swell potential of micaceous soils; method not suitable for bentonite soils.
Surface area.	Good	May overestimate the shrink-swell potential of high-activity clays mixed with sands.
Linear shrinkage.	Good	Test fairly rapid and easy to perform.

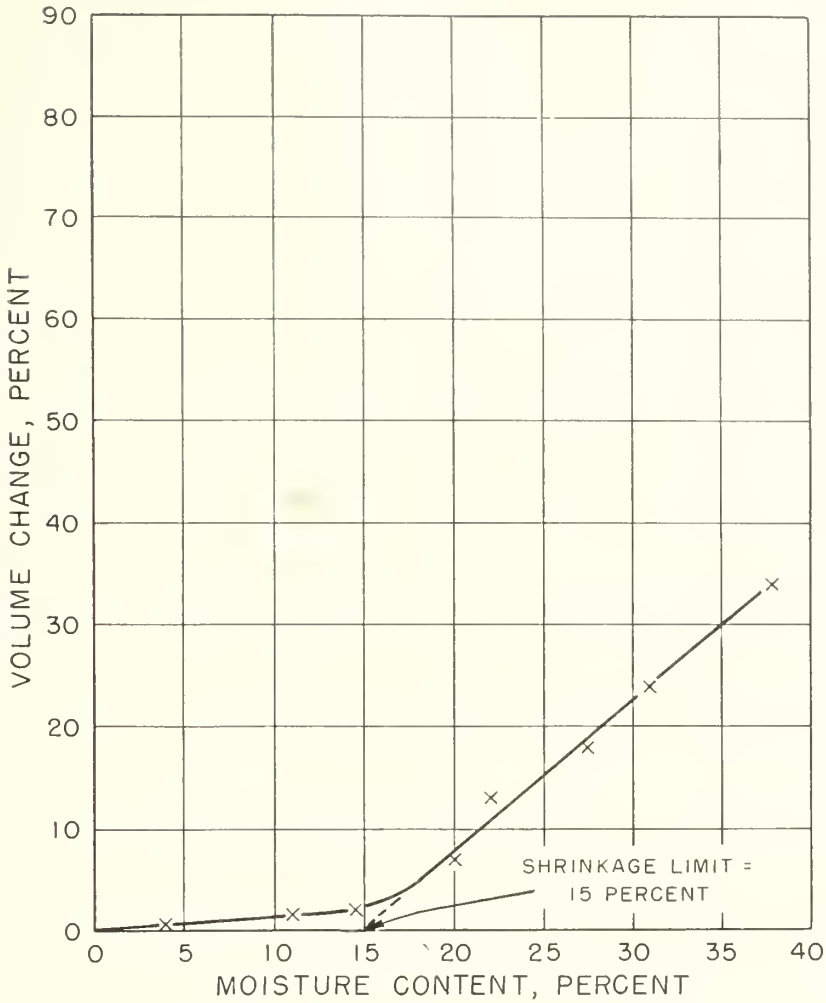

Figure 7.—Phenomenon of shrinkage limit.

of the specimens was determined during the fourth drying period. The volume of the wet specimens was calculated from their measured height and diameter; the volume of the dry specimens was measured by mercury displacement. The total volume change was expressed:

$$\text{Total vol. change} = \frac{\text{Vol. wet specimen} - \text{Vol. dry specimen}}{\text{Volume wet specimen}} \times 100$$

Tests were made on 12 soils to confirm the equilibrium hypothesis postulated from Porter's work. The texture of these soils ranged from a loessial silt having a PI of 4 (low shrink-swell potential) to a bentonite having a PI of more than 300 (very high shrink-swell potential). Test data and other characteristics of the 12 soils are shown in table 3. The volume change of the soil specimens was measured under a 0.25 p.s.i. surcharge to provide a reasonable restraint without masking volume change.

Wet-dry cyclic changes in height for two of the soils that had very different volume-change characteristics are shown in figures 4 and 5. Primary observations in this test series were: (1) The fat clay (Iredell) specimen molded at optimum moisture content swelled much more after being dried than when it was initially at optimum moisture content; (2) during the first soaking, the specimens of both soils compacted when air dry showed a change in height that was disproportionately greater than subsequent height changes; (3) in both cases, the height change of all specimens became essentially constant after several cycles of wetting and drying—at this point, the effects of the difference in initial density and moisture content appeared to have been minimized.

Because the equilibrium shrink-swell condition was achieved by the procedure described in the foregoing paragraphs, shrink-swell potential is defined as the volume change (in percent) that occurs under a 0.25 p.s.i. surcharge during the fourth drying in four cycles of wetting and drying of a soil compacted at optimum moisture content to 100 percent of the AASHO: T 99 maximum density. The shrink-swell potential for each of the 12 soils used in this study is listed in table 4.

Up to this point, the study was directed to a method of determining shrink-swell potential uninhibited by molding moisture, density, particle orientation, or the like. The method involved alternate wetting and drying of the soil and required one or more weeks to complete a test. In an effort to shorten the time required to obtain an estimate of shrink-swell potential, it was hoped that a relationship could be developed with the results obtained by some other test method that could be performed in a shorter time. For this reason, the same 12 soils were tested by 8 standard or universally accepted volume-change or swell-pressure tests, and the results were compared with the shrink-swell potential.

A method proposed by Seed, Woodward, and Lundgren (*9*), for determining swelling potential only, consists of compacting a soil to maximum density at optimum moisture content and measuring the percent of swelling after the specimen has soaked under a 1-p.s.i. surcharge. The authors concluded that for practical purposes, however, the swelling potential of a soil could be predicted from the plasticity index, but that the actual amount of swelling a soil may have in the field can only be determined from swelling tests conducted under conditions that are as similar as possible to anticipated field conditions.

In the study reported here, data obtained from shrinking and swelling tests showed that the plasticity index is also a good predictor of shrink-swell potential. In addition, the data

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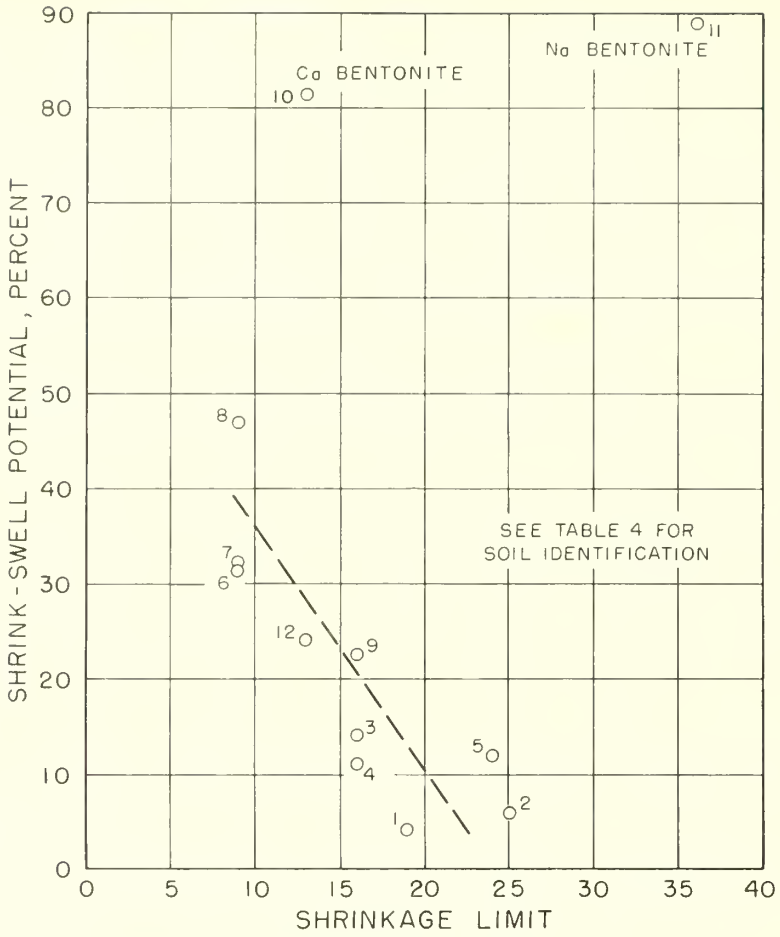

Figure 8.—Relation of shrinkage limit to shrink-swell potential.

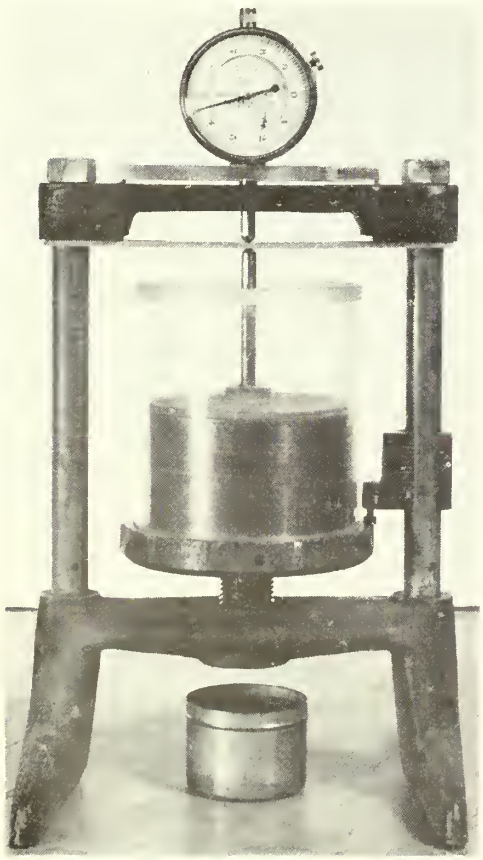

Figure 9.—Apparatus for AASHO: T 190—expansion pressure test.

compiled showed: (1) The results obtained from a variety of other types of tests on soils having a wide range of shrink-swell potential, and (2) factors affecting these test results. Limitations of some of the methods and the test results are discussed.

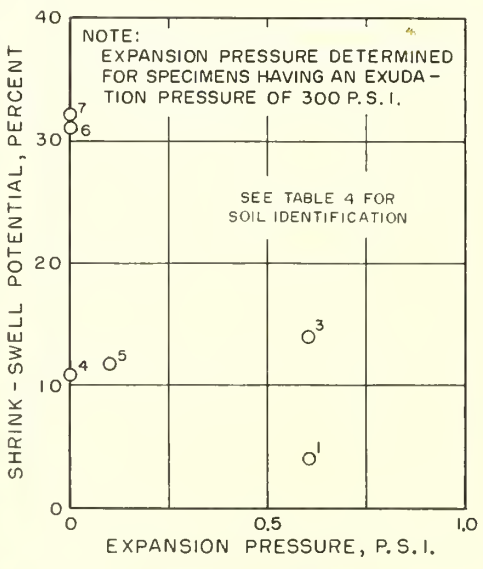

Figure 10.—Relation of expansion pressure by AASHO: T 190-61 to shrink-swell potential.

Figure 11.—Apparatus for CBR volume change test.

Test Results Compared

In the following part of this article the shrink-swell potential test results are compared to the results obtained from standard tests on the same 12 soils. In an attempt to find a rapid, reliable substitute, the standard tests were evaluated for their ability to predict the shrink-swell potential. The eight tests studied are listed as follows, with AASHO test designations where applicable: Plasticity index (PI), T 91–54; shrinkage limit (SL), T 92–60;

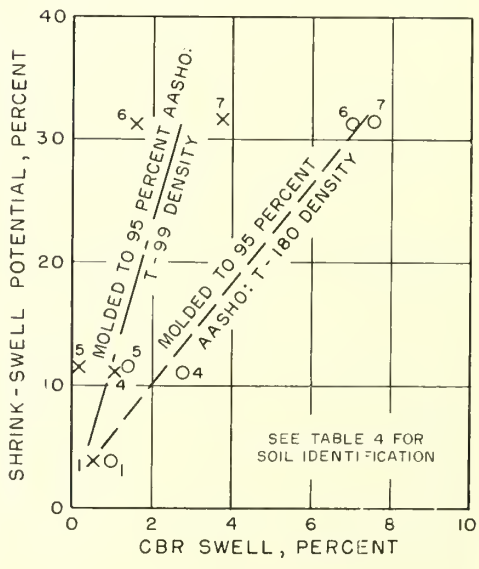

Figure 12.—Relation of CBR swell to shrink-swell potential.

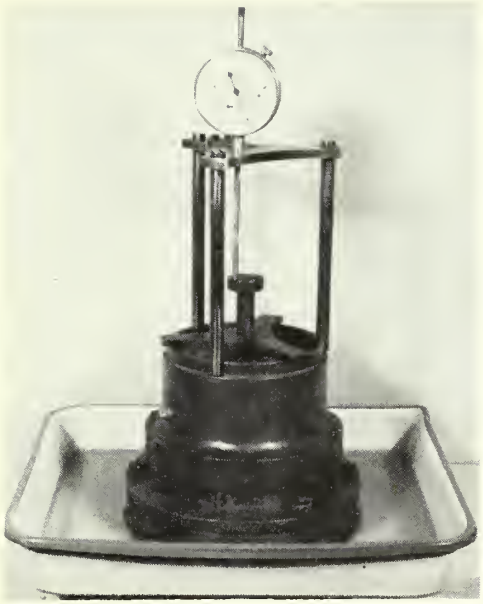

Figure 13.—Apparatus for AASHO: T 116—Volume Change Test.

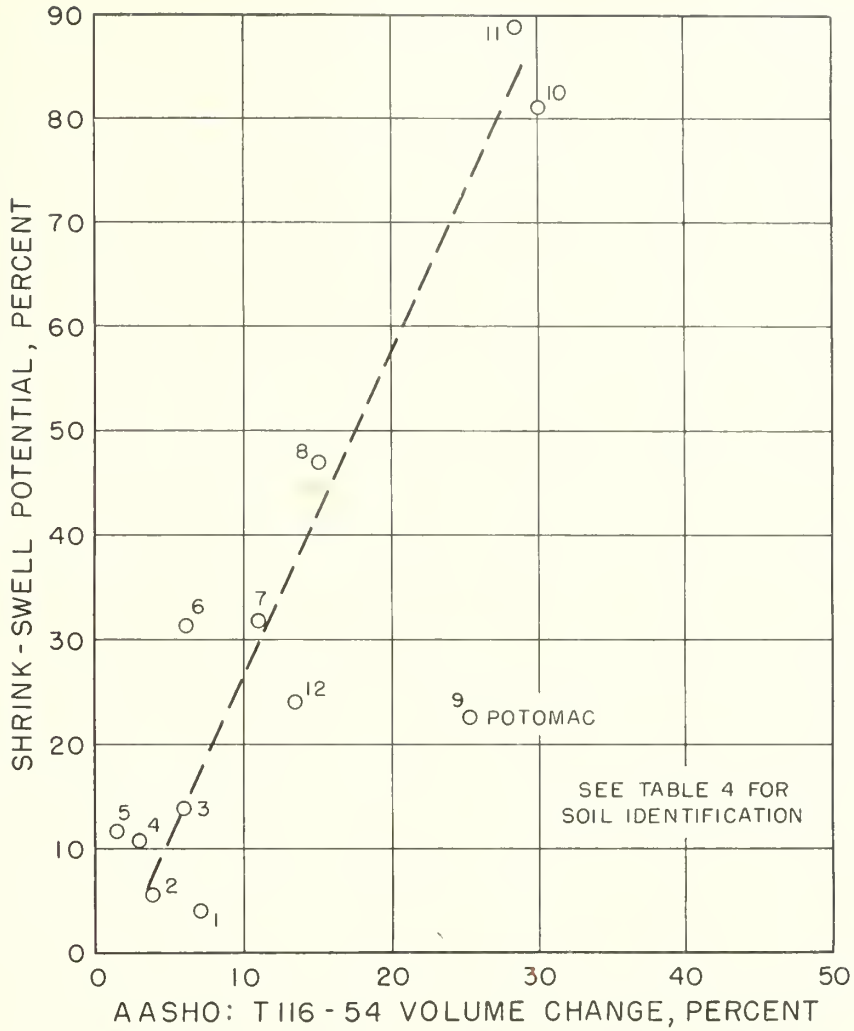

Figure 14.—Relation of AASHO: T 116-54 volume change to shrink-swell potential.

expansion pressure, T 190–61; CBR volume change, T 193–63 with modification; volume change of soils, T 116–54; Georgia volume change; total surface area; and linear shrinkage.

Plasticity index

The plasticity index (PI) is the range of moisture content over which the soil is in a plastic condition. Figure 6 shows the relationship of PI to the shrink-swell potential for the 12 soils tested. The relationship for 10 of the soils was fairly good. Based on the relationship for these 10 soils, the other 2, Cecil and Davidson, had only about one-half the shrink-swell potential that would have been anticipated on the basis of their PI's. Perhaps this was because these soils had high contents of iron oxide. The Davidson soil closely resembled lateritic soils normally developed in the tropics.

Shrinkage limit

The shrinkage limit (SL) is the calculated moisture content below which a soil has only a small change in volume as moisture content is further reduced. Figure 7 shows a plot of volume change versus moisture content; SL is indicated at the intercept of zero volume change and 15 percent moisture content. The SL of soils is generally inversely related to PI and fineness of soils. Figure 8 shows the relationship of shrink-swell potential to SL for all 12 soils. The relationship was fair, except for the two bentonites that showed much higher shrink-swell potential than would have been expected from their SL's.

AASHO: T 190

The expansion pressure test described in AASHO: T 190 measures swell pressure (fig. 9). Figure 10 shows little relationship between shrink-swell potential and expansion pressure, as determined by AASHO: T 190 procedures. In general, when these procedures are used, silty and permeable lean clay soils have medium to high swelling characteristics, and relatively impermeable fat clay soils have low to medium swelling characteristics. Specimens of fat clays tended to dry and shrink on the bottom during the soaking period. Covering the bottom of these soils with a rubber disk caused higher swelling pressures. Soaking times longer than 16 to 20 hours also caused higher swelling pressures for fat clays.

CBR Volume Changes

The CBR test measures the change in height that occurs when a compacted soil specimen is soaked, usually for 4 days (fig. 11). The soaking procedure is conducted primarily to place the soil in the weakest condition it may attain under pavements, although the measurement of height change during soaking apparently gives a fair indication of how much trouble can be anticipated because of volume change. Figure 12 shows the relationship of swelling during the CBR test to shrink-swell potential test results. For the limited data, the relationship was better for CBR specimens molded to 95 percent of the AASHO: T 180 maximum density than for those molded to 95 percent of the AASHO: T 99 maximum density.

AASHO: T 116

The test for determination of volume change of soils, AASHO: T 116, is similar to the CBR swelling procedure, except that the specimen is smaller, the soaking period is longer, and the specimens are usually compacted statically instead of dynamically. A soaking specimen is shown in figure 13. The relationships between the results of this test and shrink-swell potential are shown in figure 14. The Potomac clay soil did not appear to fit the relationship, possibly because this soil was very sensitive to loading conditions and type of compaction.

Georgia Volume Change

The Georgia test was developed to classify subgrades for road construction. Two identical specimens of each soil are compacted at optimum moisture content. One specimen is allowed to dry; the other is soaked, as shown

Figure 15.—Georgia volume change test: one specimen soaking, one specimen drying.

in figure 15. The soil classification is based on the combined volume change of the two specimens and its laboratory compacted density. The volume change of 9 of the 12 soils studied showed a fair to good relationship with shrink-swell potential (fig. 16). The Cecil soil had a different relationship, perhaps because the mica in this soil caused high swelling characteristics the first time it was wetted after being compacted. The Na and Ca bentonites could not be tested by the Georgia method because the top of the soaked samples curled and cracked as a result of the inability of the specimens to soak up water quickly enough to prevent the top of the specimens from drying.

Total Surface Area

Measurement of the total surface area is not a standard test method, but the results correlate so well with shrink-swell potential that these measurements are included here as a matter of interest. Total surface area is the sum of the external and the internal surface areas (see fig. 1, part B), expressed in square meters per gram of soil passing the No. 40 sieve. The values reported were measured by the Diamond and Kinter (*10*) glycerol retention method. The relation of total surface area to shrink-swell potential is shown in figure 17. The gradation of the Hybla Valley clay caused the shrink-swell potential to be about one-half the anticipated value based on the total surface area. Although the clay portion of this soil was chiefly montmorillonite, a material having a high surface area, there evidently was sufficient sand to form a resistance to shrinkage.

Linear Shrinkage

Linear shrinkage is the decrease in length of a bar of soil-water mixture that is dried until shrinkage ceases; it is expressed as a percentage of the original length of the bar. In addition to being a test that is fairly rapid and easy to accomplish, the dry length of the specimen at the end of the test, compared to the length of the mold, provides a physical sense of the soil's susceptibility to shrinking or swelling (see fig. 18).

For purposes of this study, linear shrinkage tests were made on soil that was slightly wetter than the liquid limit. Teflon [3] molds were used that were 20 cm. long by 2.54 cm. diameter and had a semicircular cross-section. They were lubricated with 0.3 g. petroleum jelly and the specimens were dried in an oven at 70° C. ±5°. The drying temperature was selected on the basis of results of tests indicating that linear shrinkage was affected only slightly by differences around the 70° C. This procedure produced a very good correlation of linear shrinkage with shrink-swell potential (fig. 19).

Review of Relationships

The results of the eight standard tests studied are related to shrink-swell potential with different degrees of success. Table 5 lists the general quality of the relationships and presents possible explanations as to why certain soils did not readily conform. Four of the tests—plasticity index (PI), Georgia volume change, surface area, and linear shrinkage—were closely related to shrink-swell potential. Of these four, the PI requires only a few hours to perform when rapid drying equipment is available; but the linear shrinkage requires about 16 hours (overnight drying). However, by the PI relationship, the shrink-swell potential of fine-grained but inactive clays was apparently overestimated by about two to one. Also, the PI is not so easily related to shrink-swell potential as is an actual volume change test, such as linear shrinkage. Linear shrinkage is the easiest to perform of any of the tests studied.

The relationship of linear shrinkage to shrink-swell potential was very good for the linear shrinkage test method described in this article. However, further investigations, not reported here, were devoted specifically to factors influencing the linear shrinkage test. The primary purpose of these investigations was to find a modified procedure that would provide an even better correlation. Results obtained by an alternate procedure, devised as a result of the supplementary studies, did not correlate as well with shrink-swell potential as those obtained by the test method herein described. The study, however, did show the variables and their quantitative effect on linear shrinkage test results. A limited supply of an informal report on the supplementary study is available. Interested researchers may obtain copies without cost by addressing requests to the Bureau of Public Roads, Washington, D.C., 20235, attention: Materials Division, Office of Research and Development.

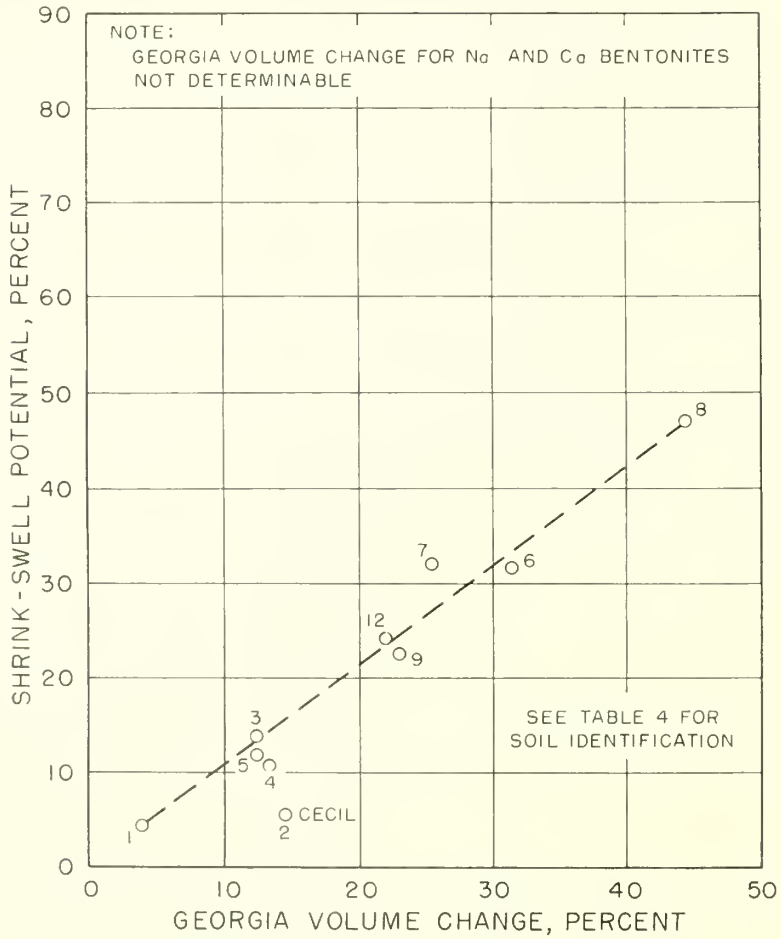

Figure 16.—Relation of volume change by Georgia method to shrink-swell potential.

[3] Trade name for polymerized tetrafluoro ethylene, a synthesized plastic.

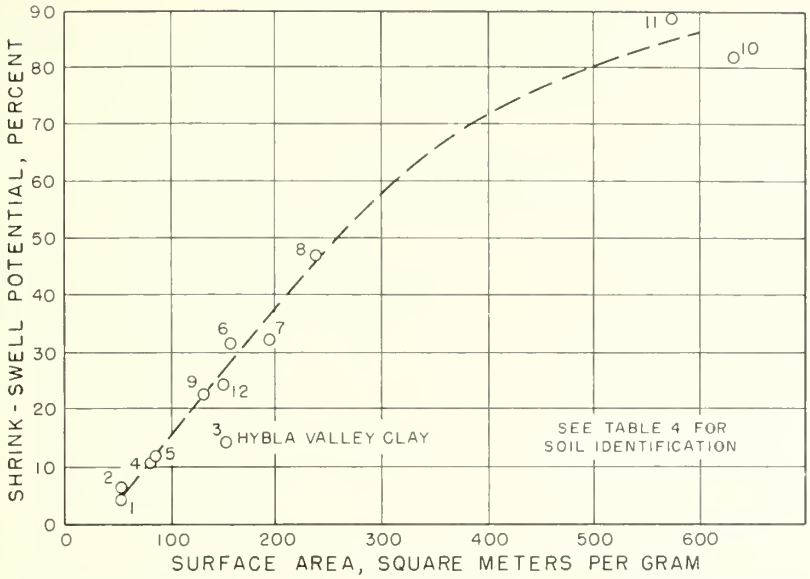
SHRINK - SWELL POTENTIAL, PERCENT
SURFACE AREA, SQUARE METERS PER GRAM
HYBLA VALLEY CLAY
SEE TABLE 4 FOR SOIL IDENTIFICATION

Figure 17.—Relation of total surface area to shrink-swell potential.

Figure 18.—Dried linear shrinkage specimens of two soils.

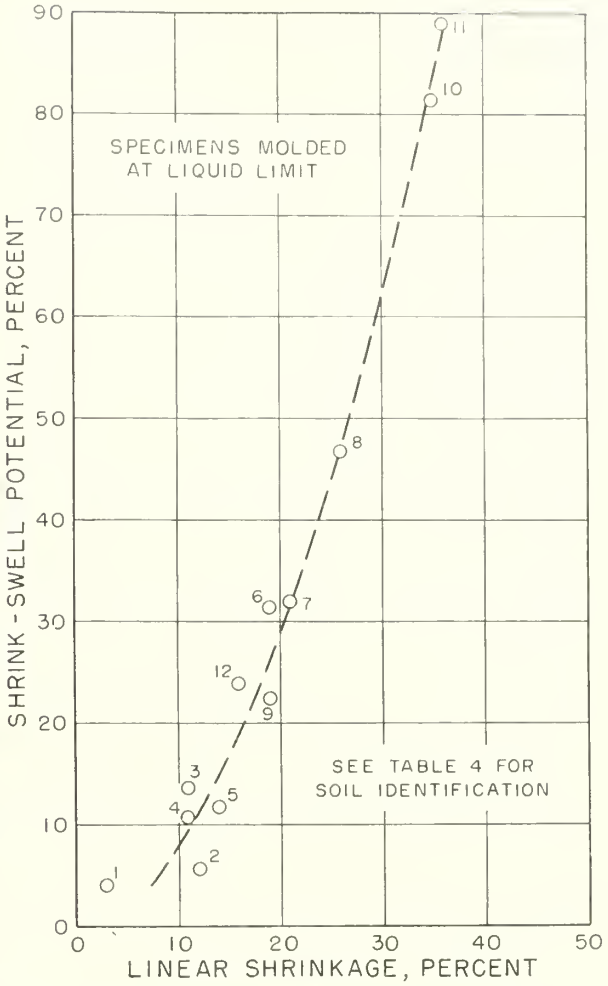
SHRINK - SWELL POTENTIAL, PERCENT
LINEAR SHRINKAGE, PERCENT
SPECIMENS MOLDED AT LIQUID LIMIT
SEE TABLE 4 FOR SOIL IDENTIFICATION

Figure 19.—Relation of linear shrinkage to shrink-swell potential.

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758-481—65——2

Friction Reducing Mediums for Rigid Pavement Subbases

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BY THE STRUCTURES AND APPLIED MECHANICS DIVISION BUREAU OF PUBLIC ROADS

Increased interest in the use of prestressed concrete pavements has been accompanied by an awareness of the need for friction reducing mediums between the subbase and the slab in order to minimize the required prestressing force. The Bureau of Public Roads has tested several different mediums in a winter-spring study and a summer study. The findings from these studies are discussed in this article.

In the studies, concrete slabs 6 feet square were moved horizontally, alternately forward and backward, several times to simulate the movement of pavements in service. The thrust necessary to cause horizontal movement and the magnitude of displacement caused by the thrust were measured from the first detectable movement until free sliding of the slab began.

All the slabs used in the test were 5 inches thick but weights were added for two sets of tests so that the effects of 8- and 11-inch slabs were obtained. Resistance to slab movement was determined for the seven underlying materials: a plastic subgrade, two types of granular subbases, and four types of mediums on a granular subbase. Most of the tests were made at a very slow rate of loading, but for comparison purposes some tests were run at medium and fast rates.

The least resistance was recorded when the medium was either a thin layer of sand or a double layer of polyethylene sheeting on a thin, leveling course of sheet asphalt. Both of these mediums were considered effective in reducing friction between the subbase and the slab.

Introduction

AS EARLY as 1924 the Bureau of Public Roads conducted studies to determine the magnitude of the resistance offered by the underlying material to the horizontal movement of concrete pavement slabs. Data from these early studies clearly showed that the resistance differed considerably according to the type of material upon which the pavement rested. Increased interest in the use of prestressed concrete pavements has been accompanied by an awareness of the need for mediums between the subbase and the slab that have a low resistance to slab movement.

For many years pavement designers have known that mediums having a low resistance to slab movement will reduce materially the direct tensile stresses induced in concrete slabs by resistance to movement during contraction. Because direct tensile stresses are generally very small for the relatively short slabs of conventional concrete pavements, little use was made of friction reducing mediums until the advent of the use of prestressed concrete pavements.

Prestressed concrete pavement slabs having lengths up to 800 feet require mediums that have a low friction coefficient for the most efficient use of the prestressing force. A medium that would reduce the frictional resistance of the subbase by 50 percent could make possible a 30- to 40-percent reduction in the required prestressing force.

Previous investigators have established that the resistance to slab movement could be decreased by different means. In 1924 Goldbeck (*1*)[2] reported that the elimination of ridges and depressions in the subgrade or the introduction of a sand layer between the subgrade and the pavement caused an appreciable decrease in the coefficient of friction. Recently, Stott (*2*), of the Road Research Laboratory of Great Britain, presented the results of a comprehensive investigation of different materials used as sliding layers over granular subbases, including polyethylene sheeting, paraffin wax, bitumen, and lubricating oil.

Heretofore, a thin layer of sand has been the most commonly used medium to reduce friction between the subbase and the slab. However, many engineers now believe that sand layers should not be used under the relatively thin prestressed highway pavements because of the possibility of aggravation of edge pumping. In recognition of the importance of friction reducing mediums for construction of prestressed concrete pavements, the Bureau of Public Roads undertook a study designed to develop comparative data on several types of mediums that have been proposed by designers of such pavements.

Conclusions

The following conclusions are based on the analysis of the data developed in the study reported here:

- For granular subbase materials, the magnitude of the coefficient of sliding friction was unaffected by (1) differences in slab thicknesses of 5, 8, or 11 inches or (2) by seasonal differences in subgrade moisture content. When the friction reducing medium was a double layer of polyethylene sheeting on a thin leveling course of sheet asphalt, the thickness of the slab did not cause any difference in the magnitude of the coefficient of friction. The effect of seasonal differences in subgrade moisture on the friction coefficient was not determined when the polyethylene sheeting was used.
- For a thin layer of sand on a granular subbase, the coefficient of friction was unaffected by rate of application of the thrusting force (rate of loading), which ranged from 1½ to 90 minutes for total applied force. The effect of rate of loading on the coefficient of friction was not determined for the other mediums used.
- The coefficient of friction for the initial movement of a slab was appreciably greater than for subsequent movements; essential stability of resistance was obtained after only 2 or 3 cycles of movement.
- Mediums of both a thin layer of sand and of a double layer of polyethylene sheeting on a thin leveling course of sheet asphalt were effective in reducing friction between the subbase and the slab.

Scope of Study

Resistance to slab movement was determined for a plastic subgrade, two types of granular subbases, and four types of mediums on a granular subbase. These underlying materials were: (1) Subgrade soil consisting of micaceous clay loam and referred to in this article as plastic soil; (2) a granular subbase consisting of material that met the

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

[2] References indicated by italic numbers in parentheses are listed on page 111.

Bureau of Public Roads grading and plasticity requirements for Federal highway projects (3); (3) a granular subbase consisting of a blend of washed sand and gravel; (4) a granular subbase, same as No. 2, and a 1-inch sand layer covered by 1-ply building paper; (5) a granular subbase, same as No. 2, and a layer of emulsified sand asphalt about 1 inch thick; (6) a granular subbase, same as No. 2, and a thin leveling course of sheet asphalt covered by a double layer of polyethylene sheeting that contained a special friction reducing additive; and (7) a granular subbase, same as No. 2, and a layer of sheet asphalt about one-half inch thick. The physical properties and AASHO classification of the subgrade and subbase materials, and information on the sheet and emulsified asphalts, are listed in table 1.

Table 1.—Physical properties of subgrade and subbase materials, and sheet asphalt and emulsified asphalt [1]

Properties	Subgrade	Granular subbase	Sand in sheet asphalt	Sand in emulsified asphalt
Mechanical analysis:				
Passing, sieve size:				
3-inch ..... percent	----	100	----	----
2-inch ..... do	----	98	----	----
1½-inch ..... do	----	96	----	----
1-inch ..... do	----	91	----	----
¾-inch ..... do	----	84	----	----
⅜-inch ..... do	100	70	----	100
No. 4 ..... do	99	61	100	95
No. 10 ..... do	98	54	98	87
No. 40 ..... do	94	36	68	37
No. 80 ..... do	----	----	30	9
No. 200 ..... do	78	15	12.5	4
Liquid limit	40	33	----	----
Plasticity index	15	16	----	----
Classification	A-6(10)	A-2-6(0)	----	----
Penetration	----	----	60-70	60-70
Asphalt content ..... percent	----	----	8.8	7.5

[1] No information was available for the subbase of blend washed sand and gravel.

For each underlying material, force-displacement curves were developed from data obtained by moving concrete slabs, 6 feet by 6 feet, horizontally, alternately forward and backward several times to simulate the behavior of pavements in service. The force or thrust necessary to cause horizontal movement of the slab and the magnitude of displacement caused by the thrust were measured from the first detectable movement until free sliding began.

The testing program was divided into a winter-spring study and a summer study, when the absorbed moisture in the subbase and subgrade were at the maximum and minimum, respectively, of the annual cycle of moisture change. All slabs were 5 inches thick. To develop data on force-displacement curves for 8- and 11-inch thick slabs, 100-pound weights were dispersed uniformly on top of the 5-inch slabs to provide the equivalent weights. A general view of the test slabs and a 5-inch slab loaded to the weight equivalency of an 11-inch slab are shown in figure 1.

Test Procedure

A schematic arrangement of the test slabs and the testing apparatus is shown in figure 2. Five wooden posts, 1-foot square, were set 3 feet deep in the ground on 9-foot centers to serve as reaction abutments for the hydraulic jack used to apply the thrusting force. The subbases and sliding mediums were placed symmetrically in 8-foot squares between the wooden posts. The 6-foot square, 5-inch thick concrete slabs were cast in place and centered in the 8-foot squares. The thrusting force was applied by a hydraulic jack to the concrete slab through a 3-foot long, 4-inch channel bearing plate. Horizontal displacement was measured by a micrometer dial on the side of the slab opposite the thrusting force.

In most of the tests the thrusting force was applied continuously for 5-minute in-

Figure 1.—Upper: General view of slabs between abutments.

Lower: A 5-inch slab loaded to the weight equivalent of an 11-inch slab.

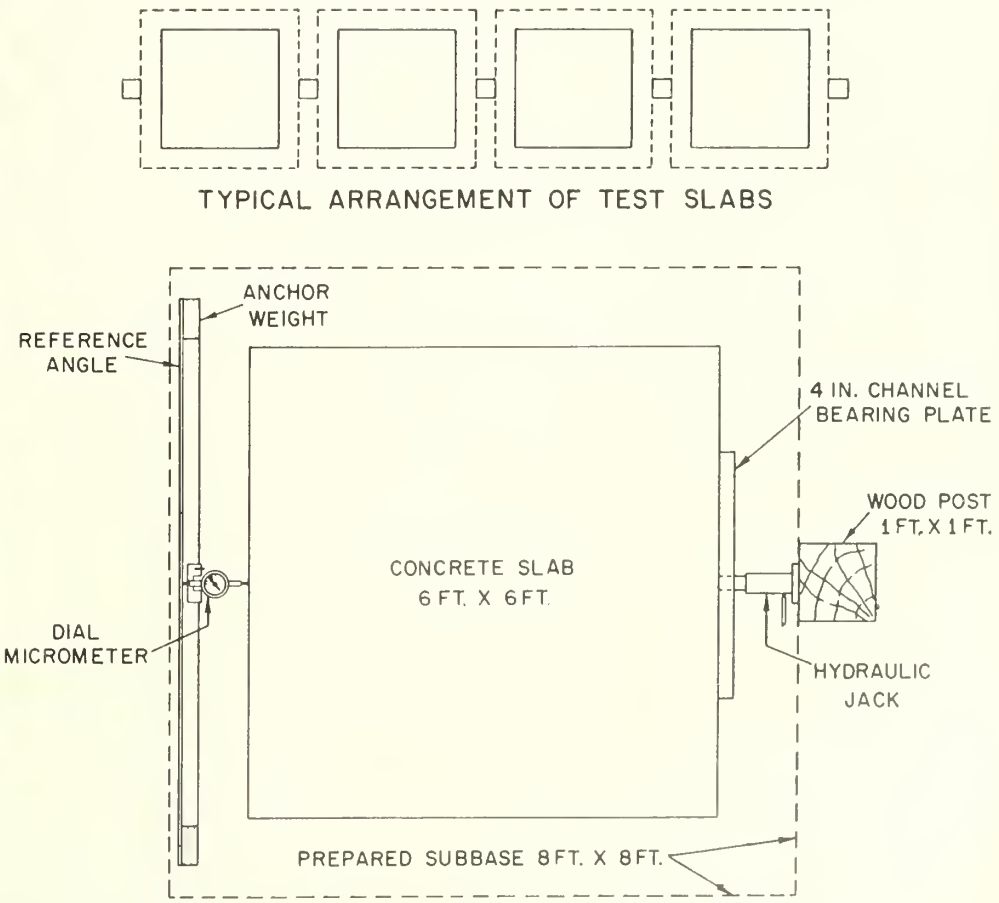
TYPICAL ARRANGEMENT OF TEST SLABS
ANCHOR WEIGHT
REFERENCE ANGLE
4 IN. CHANNEL BEARING PLATE
WOOD POST 1 FT. X 1 FT.
CONCRETE SLAB 6 FT. X 6 FT.
DIAL MICROMETER
HYDRAULIC JACK
PREPARED SUBBASE 8 FT. X 8 FT.

Figure 2.—Arrangement of test slabs and testing apparatus.

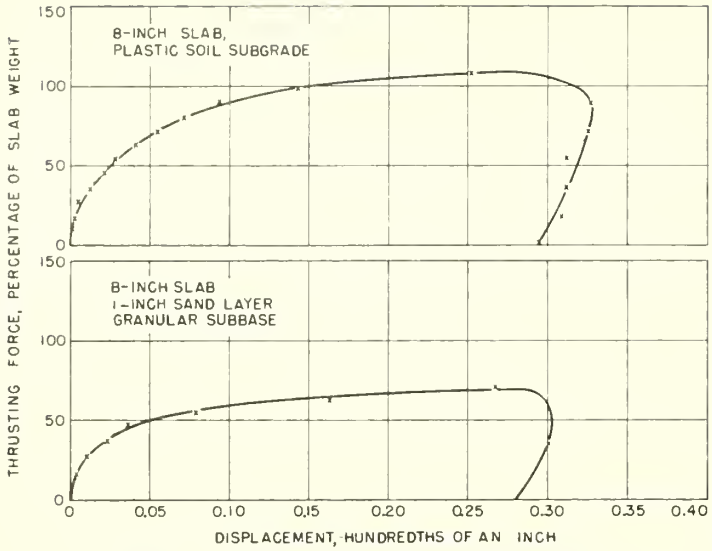

Figure 3.—Typical force-displacement curves.

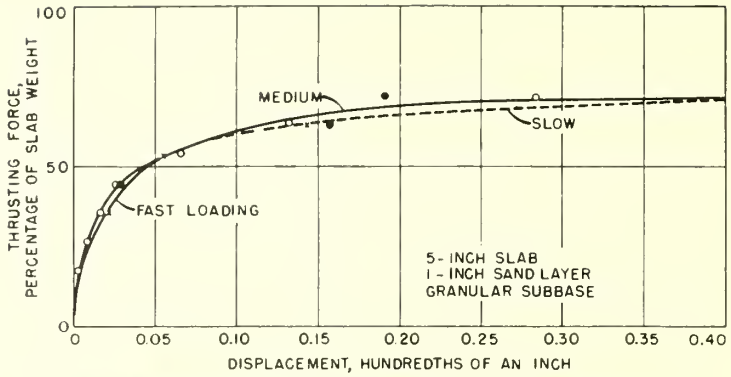

Figure 4.—Effect of rate of loading on force-displacement relations.

tervals and the load increased in increments of 200 pounds on 5-inch slabs, 320 pounds on 8-inch slabs, and 440 pounds on 11-inch slabs. The thrusting force was held constant during the 5-minute interval between the added increments. The displacement of the slab was measured immediately after the application of each increment of force and immediately before the next increment was applied. These two readings were averaged for a single displacement value. Incremental loading was continued until the slab was sliding freely and the thrusting force could not be increased.

The thrusting force was removed every 5 minutes in load decrements of 400 pounds for the 5-inch slabs, 640 pounds for the 8-inch slabs, and 880 pounds for the 11-inch slabs. When the thrusting force was released, the slabs tended to move in a reverse direction. This return movement was recorded in selected tests; it was measured immediately after the removal of each decrement of force and just before the removal of the next decrement.

Force-displacement diagrams at two additional rates of loading were also developed for the 5-inch slab on the granular subbase and sand layer. One loading rate was very fast—a force of 200 pounds was applied every 10 seconds; the other loading rate was twice as slow as the described rate—200 pounds every 10 minutes.

In general, the slabs were moved back and forth three times, a total of six instrumented runs. The slab under test was protected from the elements by a canvas shelter, which was removed when figure 1 was photographed.

Test Results

The data obtained in the testing of the slabs are shown in different ways in figures 3 to 11, inclusive. These figures illustrate certain characteristics of force-displacement behavior that have been reported previously by other investigators (*2*, *4*).

Figure 3 shows typical data developed in two of the tests. Each point on the curves represents an average for results of six displacement tests, three in a forward and three in a backward direction. As the increments of force were applied, the successive increments of displacement increased in a ratio that closely approximated a parabola. After free sliding occurred, the thrusting force could not be increased beyond the force at which the slab began to slide. The slab returned slightly toward its original position upon release of the thrusting force. This return movement was believed to have been the result of elastic deformation of the soil and, as mentioned previously, was measured in only a few of the tests because such information had little value for the purpose of this study.

Rate of Application of Thrusting Force

The effect of rate of loading on the displacement of a slab cast on the granular subbase and sand layer is shown in figure 4. The total thrusting force was applied in approximately 1½, 45, and 90 minutes for the fast, medium, and slow loading rates, respectively. Analysis of data collected showed that the rate of loading had no appreciable effect on the displacement of slabs on sand layers. This finding agrees with that of Stott (*2*) who observed that restraint offered by a friction reducing layer of sharp sand was not

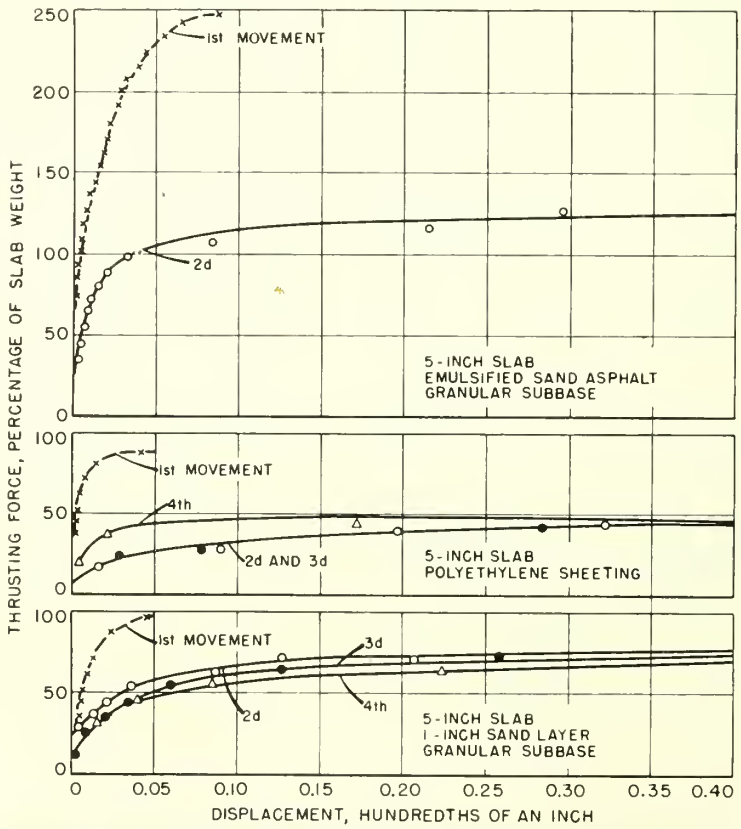

Figure 5.—Effect of successive slab movements on force-displacement relations.

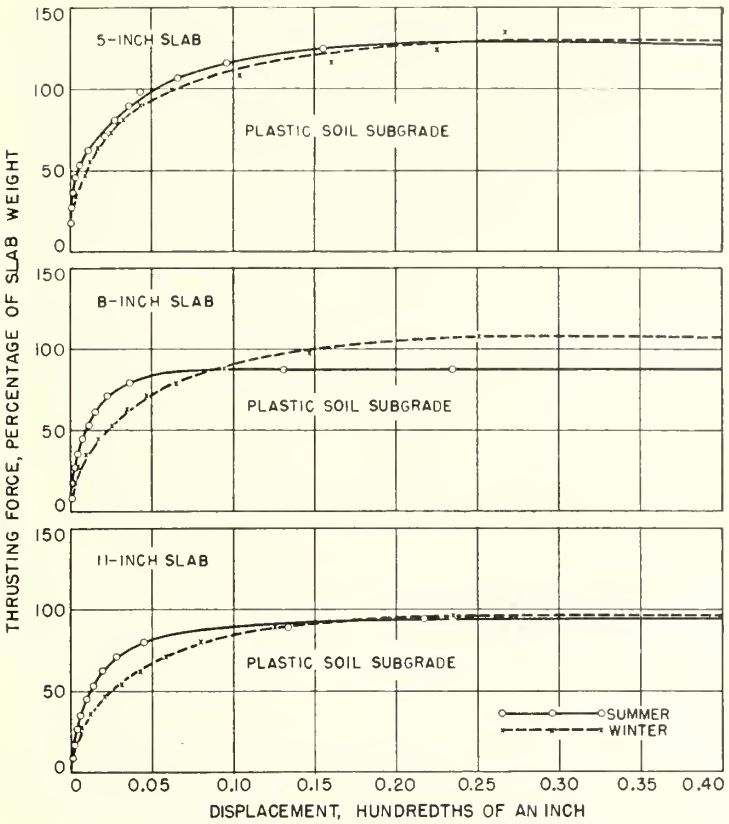

Figure 6.—Force-displacement curves for concrete slabs on plastic soil.

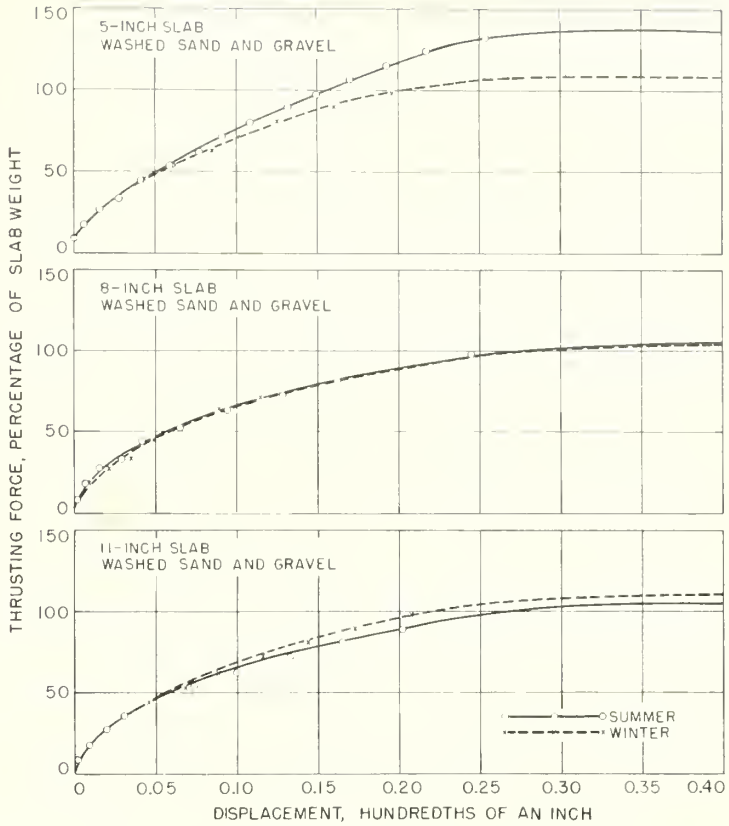

Figure 8.—Force-displacement curves for concrete slabs on blend of washed sand and gravel subbase.

markedly affected by differences in the rate of slab movement between 0.08 inch and 0.5 inch per hour.

Successive Slab Movements

As mentioned previously, the test slabs were moved back and forth several times. Examples of data obtained from these successive movements are shown in figure 5 for each of the 5-inch thick slabs cast on the sand layer, polyethylene sheeting, and emulsified sand asphalt, When free sliding occurred at first movement, the slabs moved so rapidly under the built-up thrusting force that accurate force-displacement measurements were unattainable. Therefore, plotted data on the first movement curves terminate at the point where free sliding began.

As expected, all three of the proposed friction reducing mediums produced greater resistance to slab displacement in the first movement than in the following movements (fig. 5). It is evident, however, that a condition of essential stability of resistance was obtained after only two or three movements. The data also show that the magnitude of the coefficient of friction—thrusting force at free sliding expressed in terms of percentage of slab weight—was considerably greater for the emulsified sand asphalt than for the subbases of the polyethylene sheeting and the sand layer.

Tests of the emulsified sand asphalt were discontinued after the second movement of the slab because of the large thrusting force required to cause free sliding. Likewise, tests of the sheet asphalt were concluded after the first movement of the slab because the thrusting force required to start free sliding was three times the weight of the slab. Both of these mediums bonded to the bottom of the slab; this indicated the need for use of an intermediate sliding medium beneath prestressed pavements.

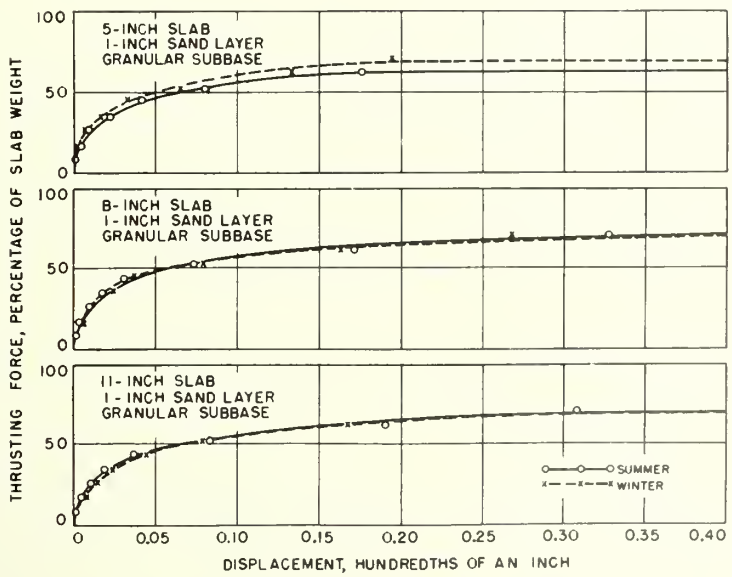

Figure 7.—Force-displacement curves for concrete slabs on granular subbase, BPR grading and plasticity requirements.

Winter-Summer Comparisons

Force-displacement relations were obtained in the summer and again in the winter-spring period for 5-, 8-, and 11-inch thick concrete slabs on the: (1) plastic subgrade soil, (2) granular subbase, (3) blend of washed sand and gravel subbase, and (4) the sand layer on a granular subbase. A 5-inch slab was cast on each of the underlying materials and, in turn, weighted to the equivalent of 8- and 11-inch slabs. The data developed in these tests are shown in figures 6 through 9. Each plotted point is the average value of displacement of five slab movements. The first movement of the slab in each of the six runs was not included in the average. Conse-

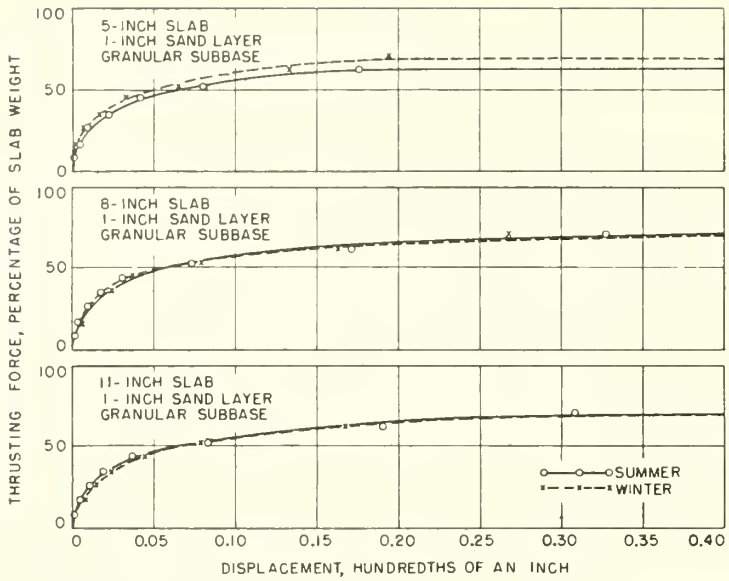

Figure 9.—Force-displacement curves, concrete slab on granular subbase, and 1-inch sand layer.

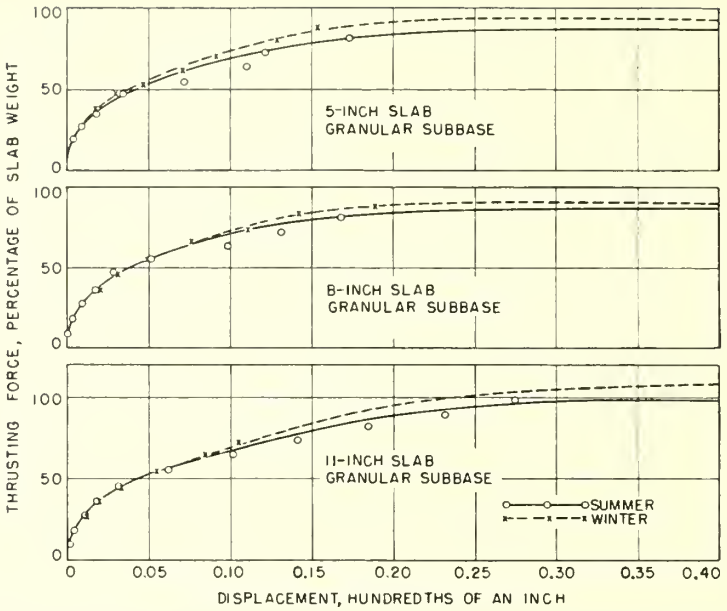

Figure 10.—Force displacement curves, leveling course of sheet asphalt covered by double layer of polyethylene sheeting.

quently, the force-displacement values represent a condition of essential stability of resistanee to slab movement. The subgrade was not frozen at the time of the winter-spring tests.

The force-displacement relations for the slabs on the plastic subgrade soil are shown in figure 6. For a displacement up to about 0.1 inch, less thrusting force was required to move the slabs on the subgrade in the winter than in the summer. However, in the free sliding range, the thrusting force necessary to move the slabs in the winter was at least equal to that required in the summer. The moisture in the top half inch of the subgrade soil was 22 and 25 percent, respectively, at the time of the summer and the winter-spring tests. The coefficient of friction tended to decrease in magnitude as the slab thickness was increased. This same tendency was observed by Teller and Sutherland (*4*), who noted that such a decrease in friction coefficient might be related to resistance to movement caused by an elastic or semielastic deformation within the soil itself.

Force-displacement relations for slabs on the granular subbase are shown in figure 7. The surface of granular subbases may differ considerably in roughness according to the type of granular material and construetion practices. In the study discussed here, the surface of the granular subbase was relatively smooth and sandy in texture. The relations recorded in the tests show that (1) the magnitude of the friction coeffieient for the granular subbase, unlike that for the plastic subgrade, tended to remain eonstant as the slab thiekness was increased, and (2) the shape of the force-displacement curve for the granular subbase, again unlike that for the plastic soil, was unaffected by moisture eonditions related to the season of the year.

The foree-displacement relations for the tests in summer and winter-spring for the slabs on the blend of washed sand and gravel and for the slabs on the 1-inch sand layer, are shown respectively in figures 8 and 9. Except for the winter data on the 5-inch slab, the force-displacement relations for the blend of washed sand and gravel were very similar to those for the granular subbase. The data, as shown in figure 9, conclusively established that the coefficient of friction for the slabs on the subbase of 1-ineh sand layers on granular subbases were unaffected by seasonal differences in subgrade moisture or by slab thicknesses. These conditions appear to be typical for slabs on granular materials.

Force-displacement relations were obtained only in the summer for the slabs on a double layer of special polyethylene sheeting over a thin leveling course of sheet asphalt. From the force-displacement relations, which are shown in figure 10, it is evident that slab thickness had little effect on the magnitnde of the coefficient of friction.

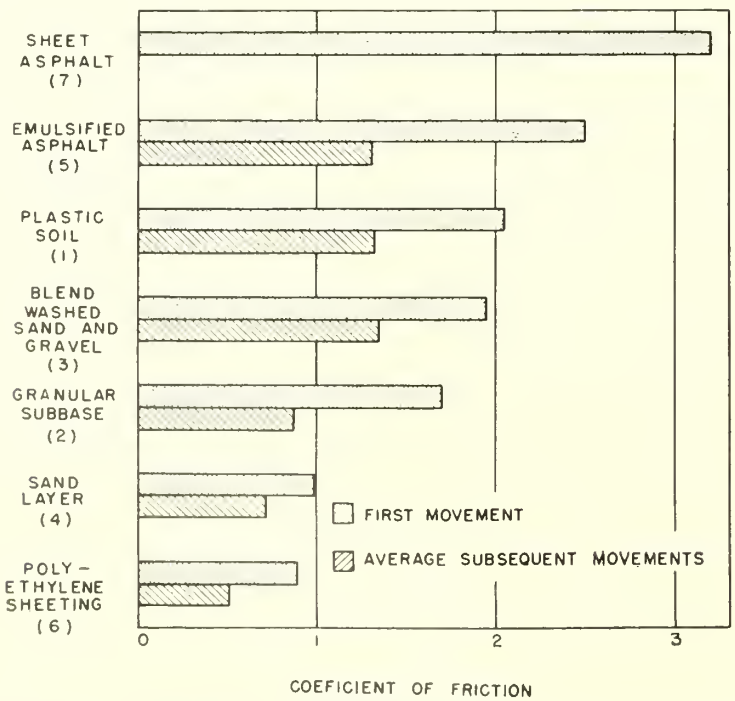

Figure 11.—Summary of coefficients of friction for slabs 5 inches thick.

Summary of Friction Coefficients for 5-Inch Slabs

Coefficient of friction data for the 5-inch thick slabs on each of the seven underlying materials studied are summarized in figure 11; the materials are arranged from top to bottom in the descending order of the magnitude of related coefficient of friction values. The numbers in parentheses identify the underlying materials with the seven listed under

Scope of Study, p. 106. These coefficients are the maximum developed in the tests at free sliding regardless of the season of the year.

Two significant facts were apparent from the test data—they are illustrated in figure 11: (1) The friction coefficient for the initial movement of a slab was appreciably greater than the coefficient for the average of subsequent movements, regardless of the underlying material. This difference in the coefficient for the first movement ranged from 35 percent greater for the slabs on the 1-inch sand medium to about 90 percent for the emulsified asphalt. (2) The lowest coefficients of friction were obtained for the polyethylene sheeting: the coefficient of friction for the first movement was 0.9, and the average for subsequent movements was 0.5. The next to lowest coefficients of friction were recorded for the 1-inch sand layer: 1.0 for the first slab movement and 0.7 for the average of subsequent movements.

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Simplified Procedures for Determining Travel Patterns Evaluated for a Small Urban Area

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

Reported by[1] CONSTANTINE BEN, Geographer, and RICHARD J. BOUCHARD,[2] and CLYDE E. SWEET, Jr., Highway Engineers, Urban Planning Division

Procedures and results obtained therewith from a two-fold research project aimed at (1) testing whether the gravity model theory would simulate travel patterns for Sioux Falls, South Dakota; and (2) evaluating simplified procedures for calibrating a gravity model for the same area are presented in this article. The application of the gravity model theory in simulating travel patterns for Sioux Falls, South Dakota, is discussed first. In 1956 a comprehensive 12.5 percent Origin-Destination survey was conducted in this small urban area, which had a population of 62,000 at the time of the survey. A three-purpose gravity model based on this full O–D survey was developed and tested. The gravity model distributions were compared with the O–D data and tests were made on trip time length frequency distributions, average trip time lengths, screenlines, and volume counts. In addition, identical tests were made on one- and six-purpose gravity models based on the same comprehensive O–D survey data.

For the second objective, to evaluate simplified procedures for calibrating a gravity model in the same small urban area, data from Sioux Falls and from several other cities were analyzed to determine the minimum acceptable sample size for use with the simplified procedures. The analyses indicated that a 599 home-interview sample might be adequate for calibration of a three-purpose gravity model. Then, the trip information available from the external cordon survey and from a subsample of 599 interviews of the original home-interview survey was used in calibrating the model. The necessary zonal production and attraction figures were determined from detailed socio-economic data by using simplified procedures. Subsequently, the synthetic gravity model distributions were checked against the data obtained from the full home-interview survey.

Although the evaluation showed that the simplified procedures were satisfactory for Sioux Falls, they may not be applicable to cities having different travel characteristics.

Introduction

A THEORY that less data on travel patterns may be sufficient for urban transportation planning than was necessary before the development of travel models provided the premise for the research reported in this article.

Since the early 1940's, the number of transportation planning studies being conducted in urban areas of the United States has been increasing. Basic data on travel patterns, social and economic characteristics of tripmakers, uses of land, and the type and extent of available transportation facilities have been collected. The interrelationships between the different kinds of data have been analyzed, and several theories of urban travel have been formulated. These theories are presented as traffic models (equations) composed of the parameters that influence the generation and distribution of urban trips, as well as the routes of the trips. One of the most widely used theories is the gravity model, which utilizes a gravitational concept to describe the distribution of trips between parts of an urban area.

Although interest has developed in the past few years (1959–63) in the use of a small sample of home-interview data for calibrating traffic models, particularly the gravity model, little has been done to test and evaluate the accuracy and validity of the theories based on this limited data. For example, in the Hartford Area Traffic Study (*1*),[3] travel data were collected from only 200, or 0.1 percent, of the dwelling units within the study area. In the Southeast Area Traffic Study (*2*), such data were collected from 1,384 or 2.0 percent, of the dwelling units within its study area. Similar sampling rates have been used in other studies (*3, 4*).

Specifically, the research described here had two main objectives: (1) to examine whether a gravity model would adequately reproduce trip distribution patterns for a particular small urban area when comprehensive O–D data were used in the calibration; and (2) to evaluate simplified procedures for calibrating a gravity model trip distribution formula for the same urban area.

In researching the first objective, full use was made of comprehensive O–D data and the resultant traffic movements calculated with the gravity model were compared with those recorded in the O–D survey.

For the second objective the trip information available from the external cordon survey and from a subsample of the original home-interview survey was used to calibrate the model. Simplified procedures were used to determine zonal productions and attractions from detailed socio-economic data. A check was made to determine whether the results obtained with the model, which was based on limited data, simulated travel patterns in the study area. This determination was made by comparing the resultant trips with comparable data obtained from the O–D survey of the same area.

The small urban area selected for the research described here was Sioux Falls, S. Dak., which had a 1956 population of

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

[2] Mr. Bouchard is presently Director, Rhode Island Statewide Comprehensive Transportation and Land Use Planning Program.

[3] The italic numbers in parentheses identify references listed on page 123.

62,000 (*5*). In that year, a comprehensive home-interview O–D survey was conducted in 12.5 percent of the nearly 20,000 dwelling units in the area. This survey was conducted at the rate recommended by the U.S. Bureau of Public Roads (*6*) for urban areas in which the population is 50,000 to 150,000. In addition to this home-interview survey, the standard external cordon and truck and taxi surveys were also conducted, as were surveys of the land use and the type and extent of the area's transportation facilities.[4] Data were available on labor force, employment, retail sales, and the results of a 1960 parking survey (*7*). The study area was divided into 74 traffic zones and 10 external stations. For summary and general analyses, these zones and stations were combined into 28 districts, as illustrated in figure 1.

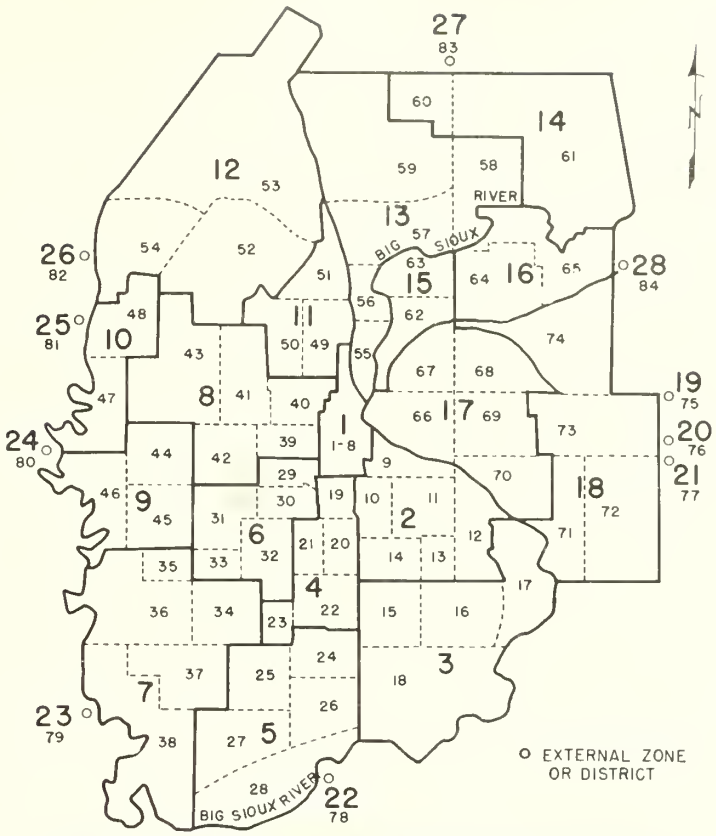

Figure 1.—Sioux Falls, S. Dak. study zones and districts.

Conclusions

Based on results obtained in the research reported herein, the following conclusions appear to be warranted:

- The gravity model formula provided an adequate framework for determining trip distribution patterns for Sioux Falls.
- A three-purpose trip stratification of home based work, nonwork, and nonhome based trips was sufficient for the small urban area.
- The synthetic procedures used in this research to compute zonal trip productions and attractions were satisfactory for this small urban area when used in combination with detailed socio-economic data and with limited travel data from a small sample survey.
- For Sioux Falls, a 600 home-interview sample used in combination with detailed socio-economic data and the standard external cordon survey provided sufficient data for a three-purpose gravity model calibration. This is a self-contained urban area having a single center and no strong travel linkages to other urban areas. The city does not have any social or economic factors that might have had a significant effect on travel patterns, and that might have required adjustments to the trip distributions as calibrated by the gravity model formula. Therefore, the findings for Sioux Falls may not apply to cities having different travel characteristics. Further research should be conducted to determine whether the findings for this small urban area can have wider application.

The Gravity Model Theory

The gravity model theory can be stated in the following described manner. The trip interchange between zones is dependent upon the relative attraction of each of the zones and upon some function of the spatial separation between zones. This function adjusts the relative attractiveness of each zone for the ability, desire, and necessity of the tripmaker to overcome spatial separation. Mathematically this theory is stated:

$$T_{ij} = \frac{P_i A_j F_{ij} K_{ij}}{\sum_{i=1}^{n} A_j F_{ij} K_{ij}}$$

Where,

T_{ij} = trips produced in zone i and attracted to zone j.
P_i = trips produced in zone i.
A_j = trips attracted to zone j.
F_{ij} = empirically derived traveltime factors (one factor for each 1-minute increment of traveltime) that are a function of spatial separation between zones. These factors reflect the average areawide effect of the spatial separation on trip interchange.
K_{ij} = a specific zone-to-zone adjustment factor to allow for the incorporation of the effect on travel patterns of social and economic linkages not otherwise accounted for in the gravity model formula.

Five separate parameters are required for calibration of the gravity model before trip interchanges can be calculated. Two of these parameters include the use of the land in the study area and the social and economic characteristics of the people who make trips; these are the number of trips produced and the number of the trips attracted. The third parameter reflects the extent and the level of service provided by transportation facilities in the area—the measure of spatial separation between zones—and is usually denoted by the minimum path traveltime between zones. The fourth parameter, the traveltime factor expresses the average areawide effect of this spatial separation upon trip interchanges between zones.

The fifth parameter in the gravity model formula is the zone-to-zone adjustment factor. It may be incorporated into the model to account for social and economic conditions and political boundaries that might have a significant effect on travel patterns, but are not otherwise accounted for in the model theory. In the study reported here, no need was developed for use of K factors.

Testing the Gravity Model Theory

A gravity model was calibrated from data obtained in the Sioux Falls O–D survey and was tested to determine whether its use would simulate the travel patterns of the O–D survey. The steps followed in this testing were identical to those documented in two recent publications by the Bureau of Public Roads (*8, 9*). Essentially these were:

- Processing basic data on the travel patterns and transportation facilities in the area, to provide three of the basic inputs to the gravity model formula; namely, zonal trip production, attraction figures, and the spatial separation between zones, as measured by traveltime.
- Developing traveltime factors to express the effect of spatial separation on trip interchange between zones.

[4] Unpublished data on the capacity and level of service characteristics of Sioux Falls transportation facilities, retail sales figures by zone, and certain employment and labor force statistics were made available to the U.S. Bureau of Public Roads by the South Dakota Department of Highways.

Table 1.—Distribution of vehicular trips, Sioux Falls, S. Dak., 1956 [1]

Purpose of trip	Trips made	Traveltime, total	Average travel-time
	Number	*Minutes*	*Minutes*
Home based work	29,882	209,128	7.00
Home based nonwork	65,759	404,749	6.15
Nonhome based	63,280	360,736	5.70
TOTAL	158,921	974,613	6.13

[1] Based on O–D survey, for 24-hour period.

• Balancing zonal attraction factors to assure that the trips attracted to each zone, as calibrated by the gravity model formula, were in close agreement with those shown in the O–D survey data.

• Examining the estimated trip interchanges to determine the need for adjustment to reflect factors not directly included in the model formula.

• Comparing the final trip interchanges obtained by using the gravity model with those obtained in the home-interview survey, to test whether the model would simulate the 1956 travel patterns in the Sioux Falls area.

For this research, the total daily vehicular trips were used that had either origins or destinations within the study area; through trips were excluded. The trips were stratified into the following listed categories: (1) home based auto-driver work trips, (2) home based auto-driver nonwork trips, and (3) nonhome based vehicular trips.

The measure of spatial separation between zones was composed of the offpeak minimum path driving time between zones plus the terminal time in the production and attraction zones connected with the trip. Terminal time was added to driving time at both ends of the trip to allow for differences in parking and walking time in the zones because of differences in congestion and available parking facilities.

Table 2.—Traveltime factor adjustment process—work trips

Driving time	Percent trips, actual	Travel-time factor No. 1	Percent trips, estimate No. 1	Adjusted travel-time factor [1]	Travel-time factor No. 2, from figure 2
1	1.68	162	1.24	219	220
2	2.93	152	2.12	210	210
3	6.09	142	4.88	177	185
4	10.28	132	10.32	131	150
5	12.61	122	13.49	114	125
6	12.57	112	13.62	103	110
7	13.91	102	13.26	107	100
8	11.22	092	11.26	92	085
9	10.91	082	11.42	78	079
10	4.20	072	6.04	50	067
11	4.40	062	5.33	51	061
12	3.98	052	3.52	59	057
13	1.53	042	1.56	41	050
14	1.34	032	1.09	39	048
15	1.70	022	0.74	51	045
16	0.04	012	0.08	06	010
17	0.01	000	0.04	00	002
18	0.00	000	0.00	00	000
19	0.00	000	0.00	00	000
20	0.00	000	0.00	00	000

[1] Derived from figures in col. 2 divided by those in col. 4 and multiplied by those in col. 3.

Basic data

Information from the home-interview, external cordon, and truck and taxi surveys was verified, coded, and punched on cards. These cards were edited to ensure that all pertinent information had been recorded correctly, and the edited cards were separated into the three trip-purpose categories. A table of zone-to-zone movements was then prepared for each trip purpose category. Each trip record was examined, and the number of trips from each zone of production to each of the other zones of attraction was accumulated. During this accumulation process, the total number of trips produced by and attracted to each zone was also determined. These zonal trip-production-and-attraction figures were subsequently used to calculate trip interchanges by using the gravity model formula.

Just as the results of the travel pattern inventories had to be processed, so did the data from the transportation facilities inventory. The processing of these data allowed the computation of the spatial separation between zones. Interzonal driving times were obtained by using a standard tree-building computer program. Intrazonal driving times were determined from an examination of the speeds on the highway facilities in each zone. Terminal times in each zone were determined by analyzing the results of the 1960 parking survey (*7*). Terminal time used for central business district zones was 3 minutes and for all other zones, 1 minute.

Traveltime factors

The best set of traveltime factors reflecting the effect of trip length on tripmaking was determined through a process of trial and adjustment. First the trip time frequency distribution was obtained for each trip purpose, by determining the number and percent of trips for each minute of minimum path driving time between all zone pairs. In table 1 this information is summarized for all three trip-purpose categories. Sets of traveltime factors were determined by drawing a line of best fit on the trip time frequency curves and by taking the traveltime factors directly from the line of best fit.

Next the gravity model formula was utilized to calculate estimated trip interchanges and trip time frequency curves using the initially estimated traveltime factors, zonal productions and attractions, and zonal separation information (traveltime). The results obtained indicated close agreement between the actual and the estimated trip time length frequency distribution curves and the average trip time lengths. However, the discrepancies between the actual and the estimated figures were larger than desired by the research staff. (The established criteria were ±3 percent difference on average trip time length, and the trip time length frequency curves closely parallel to each other.) Consequently, a revised gravity model estimate was needed.

To make a revised estimate, new sets of traveltime factors were calculated for each trip purpose category. The percentage of survey trips for each minute of driving time was divided by the percentage of trips obtained using the gravity model and the results of this division were multiplied by the initial traveltime factors, see table 2. These adjusted traveltime factors were plotted on log log paper and a smooth curve was drawn through the points as shown on figure 2.

Table 3.—Final traveltime factors by trip purpose, Sioux Falls, S. Dak.—1956

Driving time	Final travel time factors for—		
	Work trips	Nonwork trips	Nonhome based trips
1	220	280	300
2	210	260	270
3	185	220	210
4	150	160	120
5	125	130	100
6	110	090	080
7	100	085	070
8	085	070	060
9	079	060	055
10	067	050	044
11	061	039	038
12	057	035	032
13	050	027	030
14	048	025	026
15	045	021	023
16	010	016	014
17	002	000	005
18	000	000	000
19	000	000	000
20	000	000	000

The new set of traveltime factors, as represented by the smoothed curves, was subsequently used in the second calibration of the gravity model. The resultant trip time length frequency curves and average trip time length figures were again compared with the O–D data. The estimates obtained during the second calibration were within the established criteria. Consequently, the second estimate of traveltime factors was judged to adequately describe the effect of spatial separation on trip interchange between zones. These final traveltime factors are shown for each trip purpose in table 3.

Adjustment of Zonal Trip Attractions

Generally, when the gravity model formula is used to distribute the trips, the obtained zonal estimates do not equal the trips shown by the O–D surveys as actually attracted to the zones because the gravity model formula does not have any built-in adjustment to ensure such results. This difference in number of zonal attractions is a difficulty inherent in all currently available trip distribution techniques. Therefore, the number of trip attractions for each zone was adjusted to bring the number of trips distributed to a given zone into balance with the number of trip attractions of that zone, as determined by the survey.

Prior to balancing attractions, the estimated attractions obtained from calibration No. 2 were compared with the actual trips shown in the survey to determine the differences. The two items of information for each zone were plotted for each trip purpose. A technique developed by Brokke and Sosslau (*10*) was used to judge the adequacy of

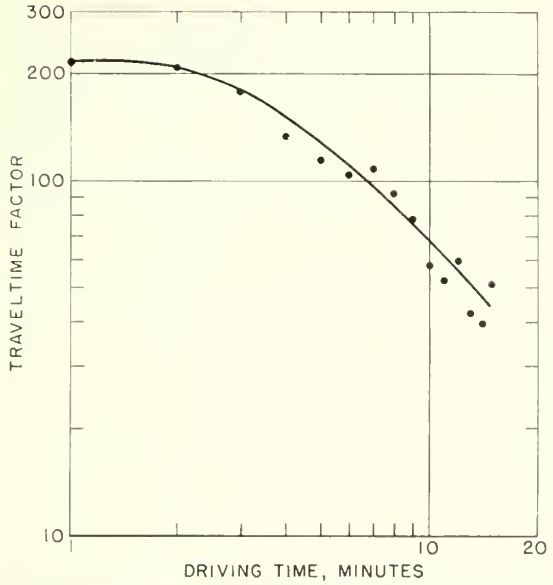

Figure 2.—Revised traveltime factors for work trips, Sioux Falls, S. Dak., 1956.

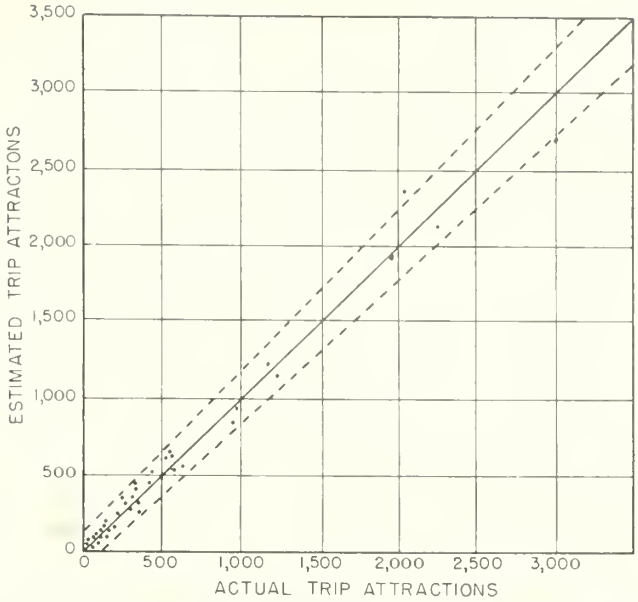

Figure 3.—Work trip attractions, calibration No. 2, Sioux Falls, S. Dak., 1956.

the estimated figures. This earlier work by Brokke and Sosslau established a reasonable approximation of the error that can be expected from O–D surveys of different sample rates, depending on the number of trips measured.

To determine the reliability (the degree of acceptability of the estimates obtained with the gravity model) of the number of trips attracted to each zone in the study area, the RMS error for each volume group for the 12.5-percent sample was plotted and the points connected by the dashed lines, as shown in figure 3. Because two-thirds of the points indicating the differences between the survey and gravity model estimated zonal trip attractions fell within these dashed lines, no adjustments were required.

Although not required, for the sake of research, the zonal attraction figures for each trip purpose were adjusted in order to obtain a more realistic measure of the error in the actual distribution of the number of trips. The adjustment was made by dividing the number of zonal trip attractions from the O–D survey by the number of trips attracted to each zone, as developed by the gravity model, and then multiplying the result by the original zonal trip attraction factor developed from the O–D survey. The amount of adjustment required for each trip purpose was relatively small—in most zones, less than 10 percent and never more than 20 percent. The required adjustment had no discernible pattern.

The interchange figures obtained through the gravity model formula were then recalculated, using the adjusted zonal attraction factors. The slight differences between calibrations No. 2 and No. 3 showed that the zonal attraction factor adjustment had very little effect on the trip volumes. This can be explained on the basis of the rather small adjustments that were required to balance the zonal adjustment factors.

Geographical bias

In using the gravity model formula, several researchers have discovered the need for adjustment factors to account for special conditions within an urban area that affect travel patterns, such as river crossings. For example, results from a study in Washington, D.C., (*11*) showed that the Potomac River crossings had some influence on trip distribution patterns. Similar findings from a study in New Orleans, La., (*12*) indicated similar problems connected with river crossings. A study in Hartford, Conn., (*1*) indicated that toll bridges crossing the Connecticut River also had a similar effect on travel patterns. In each of these cities, the effects of these conditions were adjusted by time penalties added to those portions of the transportation system for which discrepancies in the model distribution were observed.

Also, in results obtained through using the gravity model formula, some studies have indicated geographical bias that has been caused by factors other than topographical barriers. For example, in the Washington, D.C., study, adjustment factors were needed to account for the medium-income blue-collar workers residing in certain parts of the Washington area and having no job opportunities within the central parts of the city. If work trips for the Washington study had been further stratified, perhaps the need for adjustment factors would have been reduced.

Several tests were conducted on the results of calibration No. 3 to determine the need for any adjustment factors. One of these tests involved the Big Sioux River, which bisects the Sioux Falls area (fig. 1). For the number of trips crossing the Big Sioux River, the total number of trip interchanges in the O–D home-interview survey were compared directly with the number obtained by using the gravity model formula. In addition, both of these numbers were compared with actual counts taken on all the bridges crossing this river. The very close agreement between these three sources of information as shown in table 4 indicated that the Big Sioux River was not a barrier to travel.

Another test for geographical bias was conducted on trips to the central business district (CBD) of Sioux Falls. Trips from each district to the CBD, by trip purpose, as shown in calibration No. 3 of the gravity model, were compared directly with the same information from the O–D survey. Results for work trips are shown in figure 4. No significant bias was present for any of the trip purposes in the model; furthermore, the gravity model estimates were close to the figures compiled in the O–D survey.

Final Tests

The total number of trips obtained from the final calibration of the three-purpose gravity model was assigned to the transportation network, and the same assignment was

Table 4.—Comparison of vehicular trips crossing Big Sioux River, S. Dak., 1956

Facility	Vehicular trips			Synthetic gravity model estimate
	Volume count	O–D survey	Gravity model estimate	
Cherry Rock Avenue	1,511	1,640	1,660	1,512
Cliff Avenue, S.	9,132	8,420	9,444	9,208
Tenth Street	14,842	16,296	16,648	16,832
Eighth Street	8,606	6,612	6,080	6,752
Sixth Street	3,864	2,900	3,576	4,564
McClellan Street	3,069	2,596	2,032	1,972
Cliff Avenue, N.	4,699	4,156	3,904	2,048
TOTAL	45,723	42,620	43,344	42,888
Percent difference from volume count		−6.8	−5.2	−6.2
Percent difference from O–D survey	+7.3		+1.7	+0.6

Table 5.—Comparison of total trips crossing screenlines for O–D survey, and synthetic, one-, three-, and six-purpose models, Sioux Falls, S. Dak., 1956

Screenline	Trips								
	O–D survey	Synthetic model		One-purpose model		Three-purpose model		Six-purpose model	
Number	*Number*	*Number*	*Percent* [1]	*Number*	*Percent* [1]	*Number*	*Percent* [1]	*Number*	*Percent* [1]
1	7,952	7,280	−8.5	6,996	−12.0	7,344	−7.6	7,440	−6.4
2	21,012	21,120	+0.5	20,580	−2.1	20,460	−2.6	20,552	−2.2
3	13,516	13,224	−2.2	14,216	+5.2	13,900	+2.8	13,222	−2.2
4	11,384	10,428	−8.4	12,344	+8.4	12,060	+5.9	11,956	+5.0
5	9,744	8,516	−12.6	9,332	−4.2	9,252	−5.0	9,336	−4.2
6	8,784	8,440	−3.9	9,500	+8.2	9,392	+6.9	9,444	+7.5
7	6,280	6,520	+3.8	6,788	+8.1	6,824	+8.7	6,852	+9.1
8	6,568	6,100	−7.1	6,984	+6.3	7,032	+7.1	7,152	+8.9
9	2,264	1,980	−12.5	2,772	+22.4	2,676	+18.2	2,648	+17.0
10	17,448	18,420	+5.6	17,808	+2.1	17,592	+0.8	17,668	+1.3
11	5,868	4,836	−17.6	6,468	+10.2	6,532	+11.3	6,704	+14.2
12	5,592	3,872	−30.8	6,484	+16.0	6,412	+14.7	6,392	+14.3
13	13,656	15,280	+10.6	13,660	0.0	14,840	+8.7	13,924	+2.0
14	22,908	23,584	+2.9	25,096	+9.6	23,040	+0.6	22,720	−0.8
15	33,220	33,204	0.0	31,400	−5.5	32,144	−3.2	34,005	+2.4
16	10,032	10,996	+9.6	10,736	+7.0	10,012	−0.2	10,120	+0.8
17	13,424	14,220	+5.9	14,016	+4.4	13,760	+2.5	14,012	+4.4
18	9,724	12,200	+25.5	10,324	+6.2	10,276	+5.7	10,424	+7.2
19	10,060	10,720	+6.6	11,352	+12.8	11,044	+9.8	11,092	+10.3
20	5,332	5,476	+2.7	5,240	−1.7	5,420	+1.6	5,556	+4.2
21	8,496	8,364	−1.5	9,056	+6.6	9,136	+7.5	9,200	+8.3
22	13,332	14,192	+6.5	14,612	+9.6	14,504	+8.8	14,672	+10.0
23	41,500	41,468	−0.1	40,660	−2.0	41,852	+0.8	39,995	−3.6

[1] Percentage difference from O–D survey.

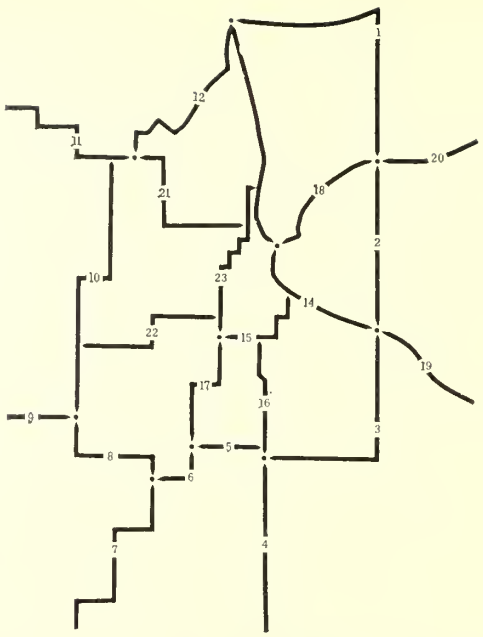

Figure 5.—Location and identification of comprehensive series of screenlines, Sioux Falls, S. Dak., 1956.

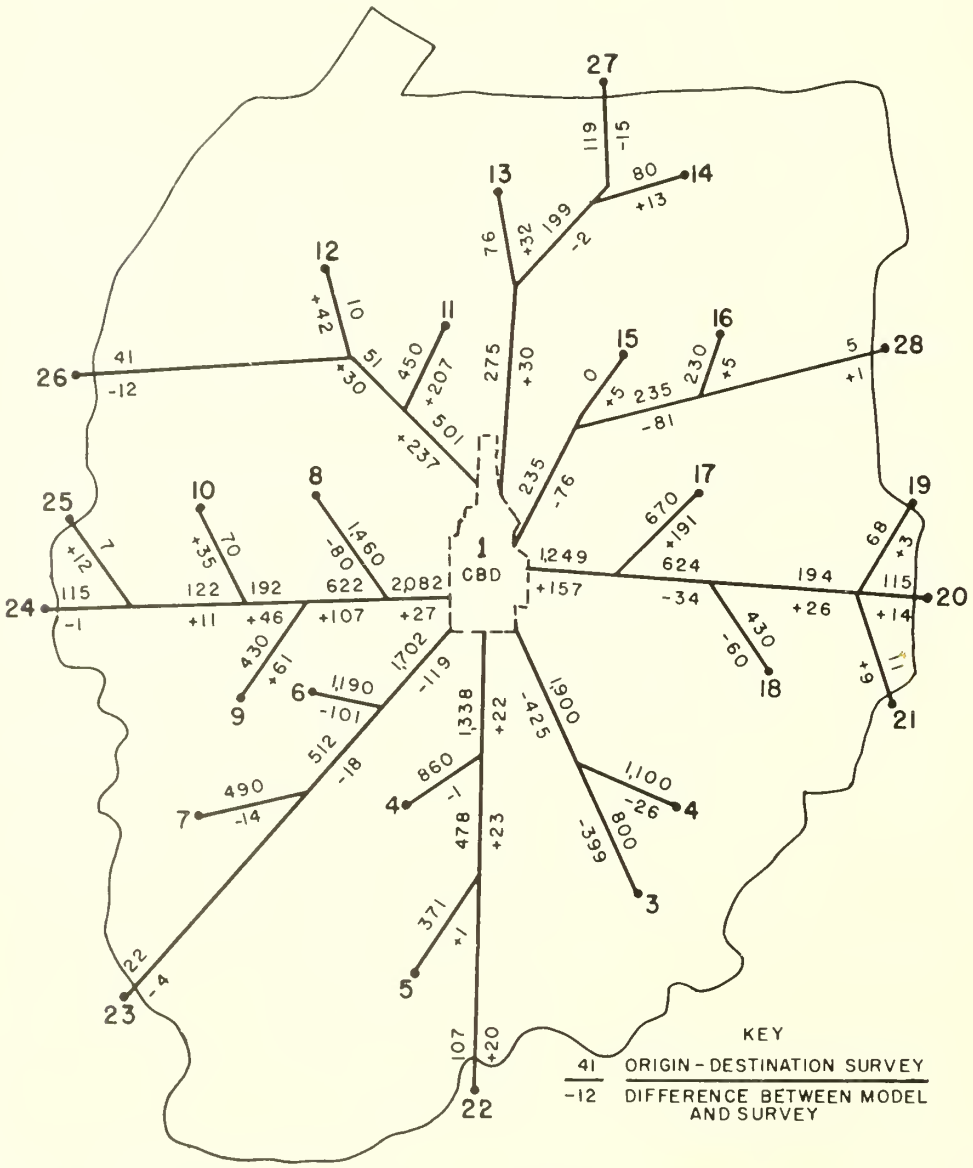

Figure 4.—Corridor analysis, actual versus estimated home based auto-driver work trips to CBD, Sioux Falls, S. Dak., 1956.

made for the total number of trips obtained from the O–D survey. An examination of the results was made by comparing the number of trips crossing a very comprehensive series of screenlines. Figure 5 shows this comprehensive series of screenlines and also identifies each. Table 5 contains tabulations on the actual and the estimated number of trips crossing each of the screen lines identified in figure 5. Only four differences between the surveyed and estimated trip volumes were larger than 10 percent but none had absolute volume discrepancies large enough to affect design considerations.

One final test was made to determine the statistical significance of the differences between the gravity model trip volume estimates and O–D survey data. The results of this test are shown in table 6. When these results were compared with the O–D survey error (*10*), the gravity model estimates had almost the same degree of reliability as the O–D survey data.

When the calibrated three-purpose gravity model was used, the results adequately simulated the trip distribution patterns of the O–D survey. Nevertheless, it is desirable to have a measure of the differences in the results that would have been obtained for lesser and higher degrees of trip stratification than the three trip purposes used in this research. To date, little has been done to investigate these differences. The results of additional analysis performed are not conclusive but do shed considerable light on the subject. In this analysis, gravity models were calibrated for the following listed trip-purpose stratifications:

- One-purpose model—total vehicular trips.
- Six-purpose model—(1) home based auto-driver work trips, (2) home based auto-driver shopping trips, (3) home based auto-driver social-recreation trips, (4) home based auto-driver miscellaneous trips, (5) nonhome based vehicular trips, and (6) truck and taxi trips.

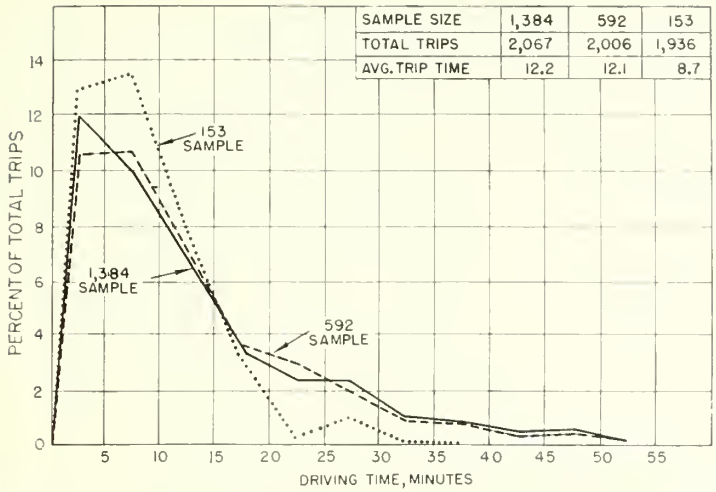
SAMPLE SIZE | 1,384 | 592 | 153
TOTAL TRIPS | 2,067 | 2,006 | 1,936
AVG. TRIP TIME | 12.2 | 12.1 | 8.7
153 SAMPLE
1,384 SAMPLE
592 SAMPLE
PERCENT OF TOTAL TRIPS
DRIVING TIME, MINUTES

Figure 6.—Trip length frequency distribution, home based auto-driver work trips, southeast area transportation study, Conn., 1960.

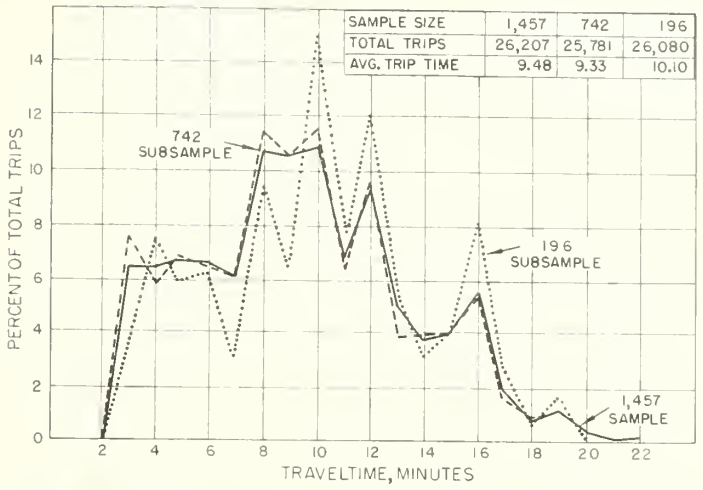
SAMPLE SIZE | 1,457 | 742 | 196
TOTAL TRIPS | 26,207 | 25,781 | 26,080
AVG. TRIP TIME | 9.48 | 9.33 | 10.10
742 SUBSAMPLE
196 SUBSAMPLE
1,457 SAMPLE
PERCENT OF TOTAL TRIPS
TRAVELTIME, MINUTES

Figure 7.—Trip length frequency distribution, home based auto-driver work trips, N.C. study, 1963.

The same techniques and the same number of calibrations were made in these two models as were made in calibrating the three-purpose model. The same tests were also performed on these models as on the three-purpose model; results had about the same degree of accuracy. The absolute and percentage differences between the number of O–D survey trips crossing the comprehensive series of screenlines (fig. 5) and those determined by using the one-purpose, three-purpose, and six-purpose calibrated models are shown in table 5. The accuracy of the three-purpose model was slightly better than that of the one-purpose model. The use of the six-purpose model, which required an extensive amount of additional effort, produced no improvement in accuracy over the three-purpose model.

Table 6.—Differences between O–D data and gravity model estimates for district-to-district auto-driver trips, for three purposes, by volume groups [1]

Trip volume	O–D survey trips		RMS error	
	HOME BASED WORK TRIPS			
Group	*Mean*	*Frequency*	*Absolute*	*Percent* [2]
0–99	21	400	17	80.95
100–199	133	40	47	35.34
200–299	259	13	87	33.59
300–499	402	13	85	21.14
500–1,499	920	8	166	18.04
	HOME BASED NONWORK TRIPS			
0–99	27	423	24	88.89
100–199	136	53	83	61.03
200–299	239	28	87	36.40
300–499	380	22	112	29.47
500–999	728	22	231	31.73
1,000–2,999	1,711	9	276	16.13
	NONHOME BASED TRIPS			
0–99	25	473	22	88.00
100–199	144	62	63	43.75
200–299	241	30	100	41.49
300–499	385	33	101	26.23
500–999	773	9	119	15.39
1,000–4,999	1,695	9	263	15.52

[1] 1956 O–D survey data versus gravity model estimates, relative difference measured in terms of percent root-mean-square error.

[2] Percent RMS error $= 100 \left(\frac{\sqrt{\frac{\Sigma(d)^2}{n}}}{\bar{x}} \right)$

Where,
d = difference between surveyed and estimated
n = number of district-to-district movements
$\bar{x}$ = mean of surveyed trips.

Investigation of Simplified Procedures

The second phase of the research was accomplished in the following steps:

- The minimum sample size of home-interview survey required to provide the information necessary to develop the gravity model for Sioux Falls was determined. This step involved the analysis of subsample data and the development of curves that could be used to determine the relative error that would occur for different size samples.
- Zonal trip production and trip attraction values for each trip purpose were estimated by utilizing the total trips from the small sample, the split among the three purposes, and certain social and economic characteristics for each zone. Zonal trip productions and trip attractions were developed by using synthetic procedures because these data are not available from small sample data.
- Gravity model trip interchanges were determined for each trip purpose by using the small sample data and the synthetic productions and attractions. The synthetic trip distribution patterns were then compared with the comprehensive O–D survey data.

Overall Travel Characteristics

According to reports on studies of other small home-interview samples, the data collected were adequate for calibrating a gravity model (*1*, *2*). The small home-interview samples provided data useful for developing the total number of trips in the area, as well as the percentage of trips for each trip purpose and travel mode category. These small samples were also reported to have provided sufficient information on the time length of urban trips, which is an important parameter in the calibration of travel models.

Some evidence is available from other reports to substantiate the findings reported in references *1* and *2*. In a recent study by the Connecticut State Highway Department,[3] subsamples of 153 and 592 home interviews were used to compare the total universe of

[3] Unpublished data on analyses made in connection with the southeast area traffic study (*2*) were made available to the Bureau of Public Roads by the Connecticut State Highway Department.

Table 7.—Comparison of total trip productions for three samples—southeast area traffic study, 1962

Trip purpose	Sample size and percent of dwelling units (sample rate)							
	1,384 2.4 percent		592 1.1 percent			153 0.3 percent		
	Expanded trips	Percent of total trips	Expanded trips	Percent of total trips	Difference [1]	Expanded trips	Percent of total trips	Difference [1]
	Number	*Percent*	*Number*	*Percent*	*Percent*	*Number*	*Percent*	*Percent*
Home based work	2,067	32.9	2,006	30.9	−3.0	1,936	32.6	−6.3
Other home based	3,218	51.3	3,446	53.1	+7.1	3,139	52.9	−2.5
Nonhome based	990	15.8	1,040	16.0	+5.1	859	14.5	−13.2

[1] Percentage difference from sample of 1,384.

Table 8.—Comparison of total trip productions for six samples—N.C. research project, 1963

Trip purpose	Sample size																	
	1,457		742			196			383			248			192			
	Expanded trips	Percent of total trips	Expanded trips	Percent of total trips	Difference [1]	Expanded trips	Percent of total trips	Difference [1]	Expanded trips	Percent of total trips	Difference [1]	Expanded trips	Percent of total trips	Difference [1]	Expanded trips	Percent of total trips	Difference [1]	
	Number	*Percent*	*Number*	*Percent*	*Percent*	*Number*	*Percent*	*Percent*	*Number*	*Percent*	*Percent*	*Number*	*Percent*	*Percent*	*Number*	*Percent*	*Percent*	
Home based work	26,207	38.9	25,781	38.6	−1.6	26,089	39.0	−0.5	24,382	38.5	−7.0	25,920	40.0	−1.1	27,498	39.3	+4.9	
Other home based	27,760	41.2	27,887	41.7	+0.5	27,101	40.5	−2.4	27,983	44.2	+0.8	27,896	43.0	+0.5	26,637	38.1	−4.0	
Nonhome based	13,437	19.9	13,194	19.7	−1.8	13,720	20.5	+2.1	10,991	17.3	−18.2	11,053	17.0	−17.7	15,802	22.6	+17.6	

[1] Percentage difference from sample of 1,457.

Table 9.—Comparison of total trip productions for three samples—Pittsburgh, Pa., 1958

Trip purpose	Sample size and percent of dwelling units (sample rate)							
	16,169 4.0 percent		2,021 0.5 percent			404 0.1 percent		
	Expanded trips	Percent of total trips	Expanded trips	Percent of total trips	Difference [1]	Expanded trips	Percent of total trips	Difference [1]
	Number	*Percent*	*Number*	*Percent*	*Percent*	*Number*	*Percent*	*Percent*
Home based work	796,195	34.1	792,576	33.9	−0.5	765,480	33.3	−3.9
Home based other	425,074	18.2	440,784	18.8	3.7	436,920	19.0	2.8
Home based soc-rec	288,047	12.3	293,752	12.6	2.0	311,280	13.5	8.1
Home based shop	286,883	12.3	276,416	11.8	−3.6	289,640	12.6	1.0
Home based school	232,875	10.0	218,264	9.3	−6.3	191,920	8.4	−17.6
Nonhome based	306,915	13.1	318,688	13.6	3.8	303,520	13.2	−1.1

[1] Percentage difference from 4 percent sample.

Table 10.—Comparison of total trip productions for three samples [1]—Sioux Falls, S. Dak., 1956

Trip purpose	Sample size and percent of dwelling units (sample rate)							
	2,399 12.5 percent		599 3.1 percent			199 1.0 percent		
	Expanded trips	Percent of total trips	Expanded trips	Percent of total trips	Difference [2]	Expanded trips	Percent of total trips	Difference [2]
	Number	*Percent*	*Number*	*Percent*	*Percent*	*Number*	*Percent*	*Percent*
Home based work	25,161	24.2	26,564	24.4	5.6	26,292	26.4	4.5
Other home based	50,782	48.9	53,848	49.4	6.0	47,232	47.4	−7.0
Nonhome based	27,924	26.9	28,516	26.2	2.1	26,040	26.2	−6.8

[1] These figures are from home-interview person trip data only; they do not include information available from the external cordon survey. Auto-driver trip data from both of these sources of data were used in developing trip interchanges synthetically.

[2] Percentage difference from 12.5 percent sample.

trips, as well as the trip time length frequencies for each of three trip purposes. The subsamples were taken from an original home-interview sample of 1,384, in itself a small sample and one that contained inherent sampling error. Some of the results are shown in table 7. This study showed that a sample of 592 home interviews produced approximately the same results for the total number of trips, by trip purpose, as the use of the 1,384 interviews. The trip time length frequency distributions and average trip time lengths for each trip purpose and sample size were also compared and the results showed that the trip time length frequency distributions and mean trip time lengths were very similar for the sample sizes of 1,384 and 592. An example for work trips is shown in figure 6.

Further verification of the procedures used in the two Connecticut studies came from a study made in North Carolina (*13*). The total trips and trip percentages for three trip purposes for subsamples of 192, 196, 248, 383, and 742 were compared. These subsamples were drawn from an original sample of 1,457 home interviews in Fayetteville, N.C. (population 47,106). Some of the results reported were: (1) samples as small as 742 interviews might produce approximately the same data for total trips by purpose as the larger sample of 1,457 interviews (table 8), (2) trip time length frequency distributions and mean trip times were similar for the 742 sample and the 1,457 sample (see figure 7), and (3) a sample of more than 383 interviews is necessary for adequate reproduction of mean trip times.

Recently the Urban Planning Division of the Bureau of Public Roads made a similar study of the differences in total number of trips, purpose split, average trip time lengths, and trip time length frequency distribution for subsamples of 2,021 and 404 home interviews from a total sample of 16,169 interviews.[6] The results of the comparisons of the total number of trips and purpose for each subsample are shown in table 9. Trip time length figures for each of the six purposes in each of the sample sizes tested showed that these small samples yielded adequate data on these *overall* travel characteristics, results for work trips are shown in figure 8.

The minimum acceptable sample size for the Pittsburgh study, however, was about 2,000 interviews, a sample considerably larger than the minimum acceptable sample size from the Connecticut and North Carolina studies. This larger minimum acceptable sample was not surprising in that person trips for six trip purposes were analyzed for this major urban area. The smaller Connecticut and North Carolina studies required the analysis of only auto-driver trips for three trip purposes. A six-purpose stratification requires a larger sample size to develop estimates of the same accuracy because of the reduced number of trips in each purpose.

Several subsamples of the Sioux Falls home-interview data were also examined for accuracy of figures on total number of trip productions, average trip time lengths, and trip time length frequency distributions by trip purpose. The results of these analyses for 599 and 199 dwelling-unit subsamples and the Sioux Falls O–D 2,399 sample are shown in table 10 and figures 9, 10, and 11. These results reinforced the findings of the previously mentioned studies, which indicated that samples as small in number as 600 might be used to determine the *overall* average characteristics of travel in this

[6] Unpublished summary of procedures and results of an analytical study of subsamples in which data from the 1958 Pittsburgh Area Transportation Study were used.

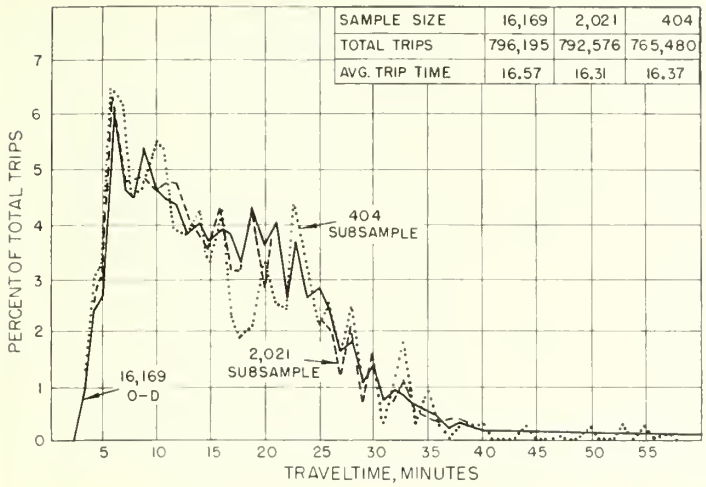

Figure 8.—Trip length frequency distributions, home based person work trips, Pittsburgh, Pa., 1958.

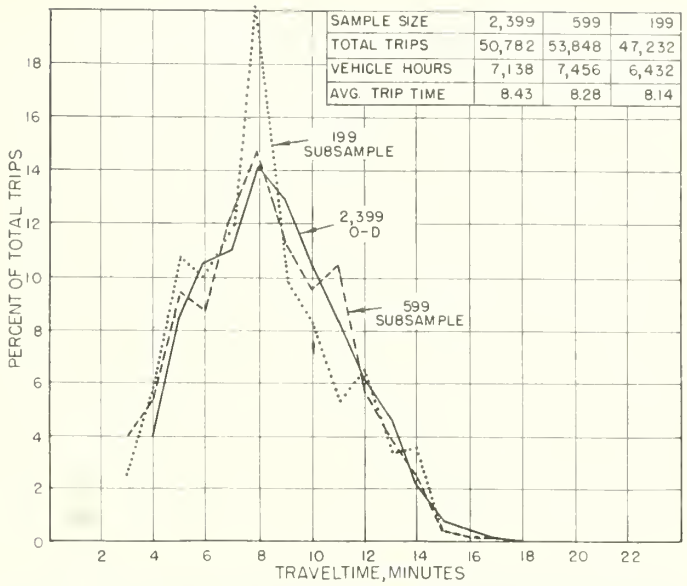

Figure 10.—Trip length frequency distributions, home based nonwork trips, Sioux Falls, S. Dak., 1956.

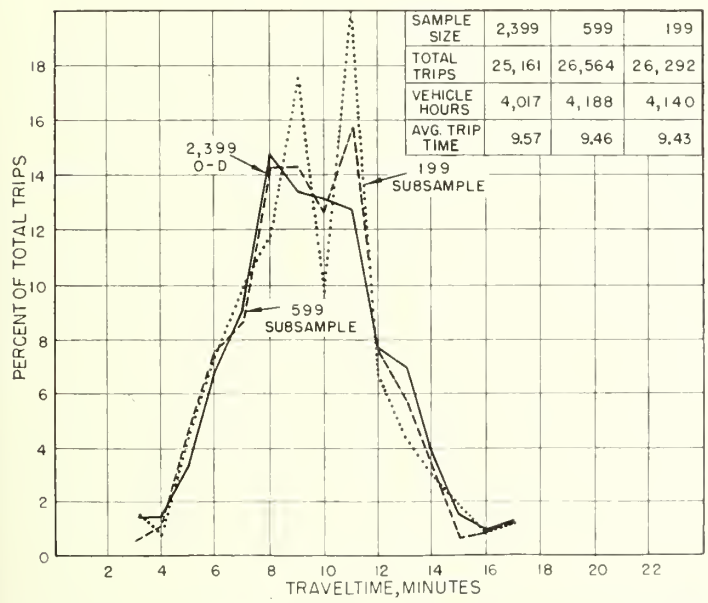

Figure 9.—Trip length frequency distributions, home based auto-driver work trips, Sioux Falls, S. Dak., 1956.

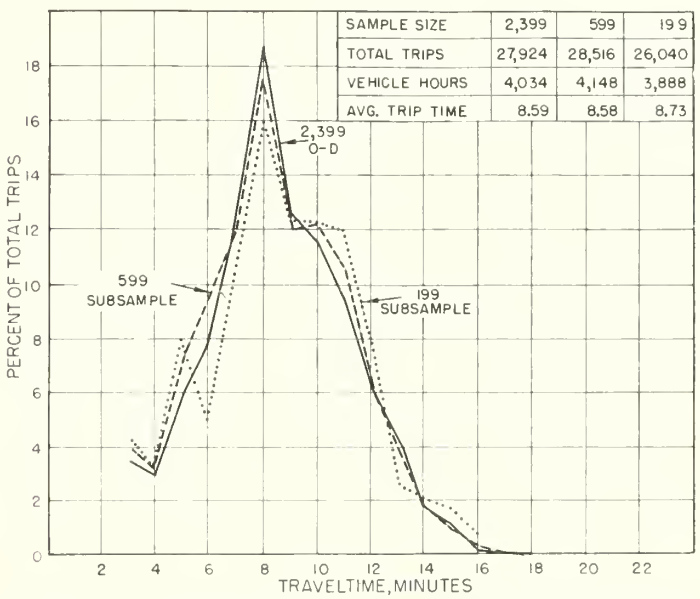

Figure 11.—Trip length frequency distributions, nonhome based trips, Sioux Falls, S. Dak., 1956.

small urban area when only three trip stratifications are used.

The results of the tests made with the Sioux Falls subsamples were analyzed to determine whether general curves could be developed to approximate the error that would occur in mean trip time length, total trips by trip purpose, and trips per dwelling unit for different sample sizes. The curves were developed and are shown in figures 12 and 13. They show for Sioux Falls the error that could be expected to occur in the indicated parameters for different sample sizes, based on the known variance in the trip data. They were developed from the relationship between the standard deviation of the mean and the square root of the sample size.

A statistical analysis of small samples of the Pittsburgh, Pennsylvania, trip data was made to determine the reliability of small samples for making estimates of trip production and average trip time length characteristics. The results of this analysis (figs. 14 and 15) show the reliability of small sample home interview surveys in determining the overall travel characteristics of an urban area.

Zonal Trip Productions and Attractions

The part of the research discussed in the following paragraphs was based entirely on the sample-size analyses. The results of the 599-interview subsample of the Sioux Falls O–D home-interview survey and the standard external cordon survey were combined with simplified estimating procedures and used in calibrating a synthetic gravity model.

As previously stated, two of the basic parameters required to estimate trip interchanges by using the gravity model formula are the number of trips produced in each zone and the number of trips attracted to each zone for each trip purpose category. Because this information cannot be obtained directly from a small sample home interview, some assumption must be made as to how the total number of trip productions and trip attractions are distributed on a zonal basis. The assumptions made and procedures used to obtain zonal trip production and attraction figures in this research were similar to synthetic procedures that have been previously reported (*4, 14*). In these procedures detailed socio-economic data are used to develop estimates of productions and attractions for use with the gravity model trip distribution technique. For example, data on the labor force was used in developing work trip production figures; employment, for work trip attractions; and retail sales, for nonwork trip attractions.

Table 11.—Vehicular trip productions and attractions, by trip purpose, Sioux Falls, S. Dak., 1956

Trip Purpose	Trips per car[1]	Productions			Attractions		
		Internal[2]	External[3]	Total	Internal[2]	External[3]	Total
Home based work	1.36	27,475	2,175	29,650	28,212	1,438	29,650
Home based nonwork	2.84	57,219	8,010	65,229	60,123	5,106	65,229
Nonhome based	2.98	59,966	4,956	64,922	59,847	5,075	64,922
Total vehicular	7.18	144,660	15,141	159,801	148,182	11,619	159,801

[1] Information includes travel data from both the 600 home-interview sample and the truck and taxi surveys.
[2] These figures were obtained by multiplying the trip rates by the total cars owned by the residents of the study area
[3] These figures are from the standard external cordon survey.

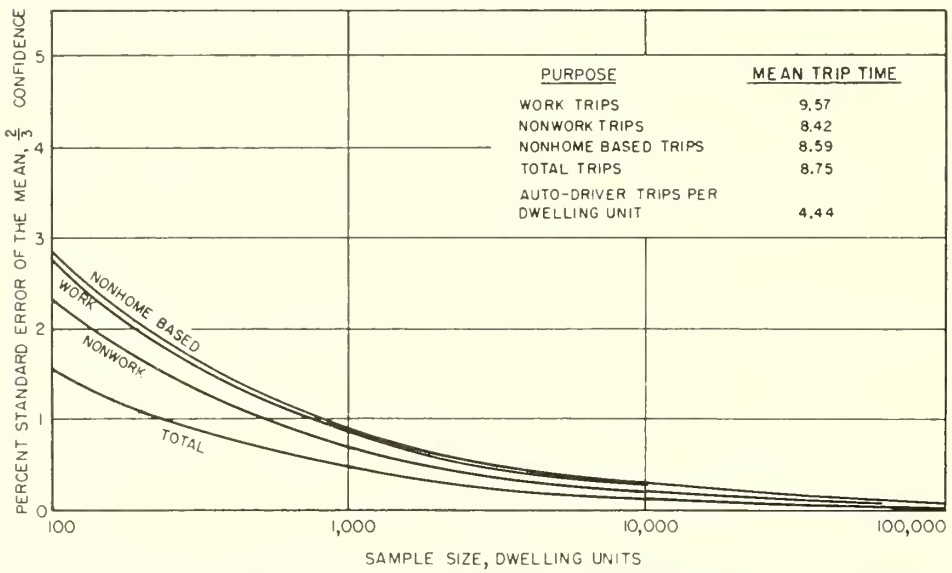

Figure 12.—Percent standard error of mean trip time versus sample size in dwelling units (log scale), Sioux Falls, S. Dak., 1956.

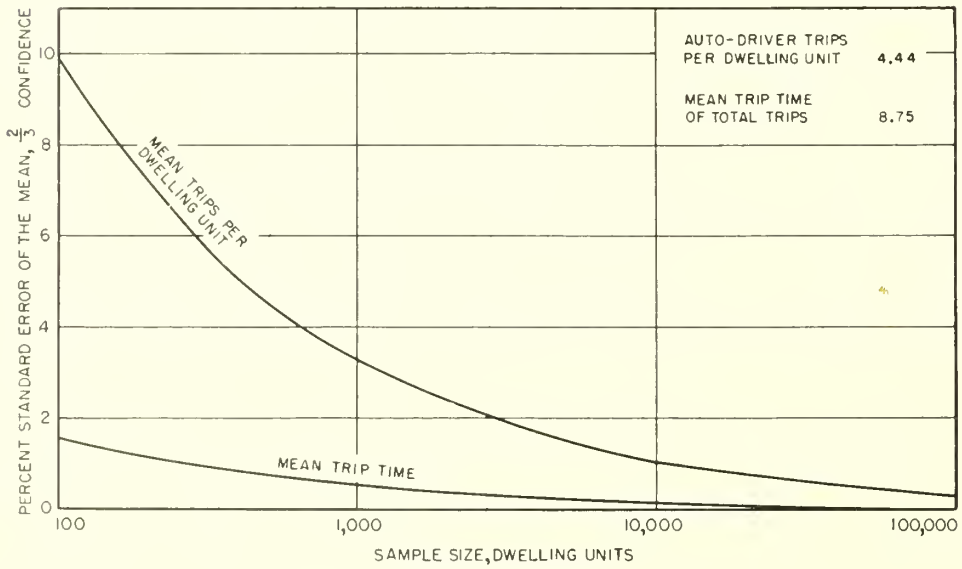

Figure 13.—Percent standard error versus sample size in dwelling units (log scale) for mean trip time and trips per dwelling unit, Sioux Falls, S. Dak., 1956.

The average number of trips made in each car owned by the persons interviewed totaled 7.18 (table 11); 1.36 were work trips, 2.84 were nonwork trips, and 2.98 were nonhome based trips. By applying these rates to the total number of automobiles in the area, a total number of trips, by trip purpose, was obtained. The total number of automobiles for any study area can be obtained from several sources, such as census data (only for the census year), State, county, or city auto registration records, or special surveys. For the study reported here, the information was obtained from a comprehensive home-interview O–D survey (1956, Sioux Falls, S. Dak.). The resultant estimates of total trip production for each trip purpose are shown in table 11. As total trip productions for the entire study area must equal total trip attractions in the entire study area for each trip purpose category, estimates of total trip attractions were also made and are shown in table 11.

Table 12.—Percentage of all work trips made by transit[1]

Cars per 1,000 persons	Net land per family—in sq. feet					
	10,000	5,000	2,500	1,200	600	300
500	5	7	11	19	33	65
450	7	9	13	21	35	67
400	9	11	15	23	37	69
350	11	13	17	25	39	71
300	13	15	19	27	41	73
275	14	16	20	28	42	74
250	15	17	21	29	43	75
225	16	18	22	30	44	76
200	17	19	23	31	45	77
175	18	20	24	32	46	78
150	19	21	25	33	47	79
125	20	22	26	34	48	80

[1] *From Use of Mathematical Models in Estimating Travel*, by Alan M. Voorhees, Journal of the Highway Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HW4, December 1959, pp. 131–132.

Home Based Auto-Driver Work Trips

Zonal production and attraction figures for home based auto-driver work trips were derived chiefly from employment and labor force statistics.

Zonal trip productions

Zonal work trip productions in the 74 internal zones were derived from zonal labor force information. Labor force data are generally available from census reports, labor statistics, and labor reports. In the research reported here, the information for each zone was taken from data available for Sioux Falls.[7] Information reported in studies made in other areas (*4, 14*) is that there are about 0.80 daily one-way work trips produced by each person in the labor force. This figure is not 1.0 work trips per person because some persons in the labor force are unemployed, on vacation, or walk to work, etc. Trip patterns shown in the O–D survey data in Sioux Falls were similar. Therefore, to determine the total number of work trip productions in each zone, the number of persons in the labor force in each internal zone was multiplied by 0.80.

To determine transit usage the zonal information on car ownership and net residential density was used. An index of transit usage, shown in table 12, was developed from Chicago, Ill., O–D survey data. By using the data from this table, the zonal indexes of transit usage were obtained. The resultant zonal indexes were then totaled and equated to the work trip transit usage for Sioux Falls as determined from the small sample of the

[7] See footnote 4, page 113.

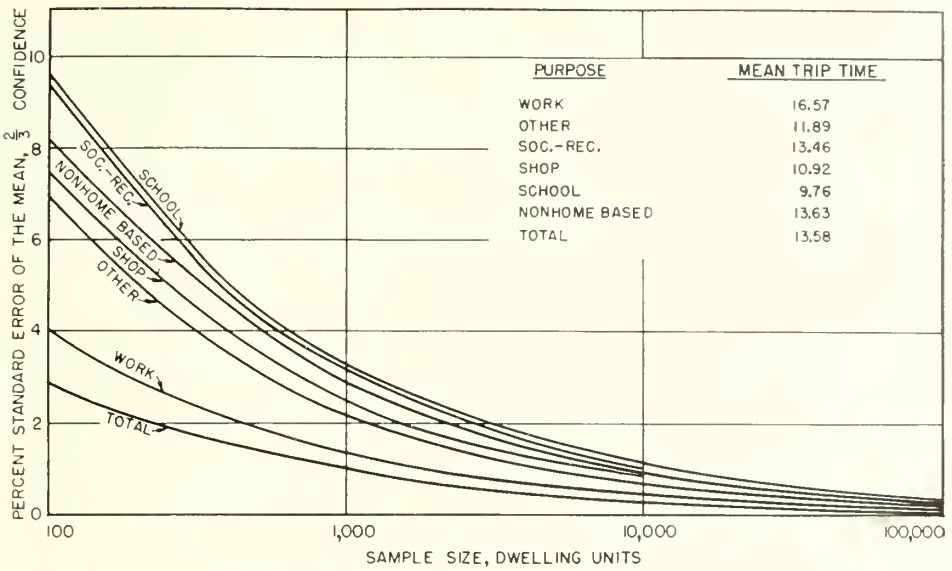

Figure 14.—Percent standard error of mean trip time versus sample size in dwelling units (log scale), Pittsburgh, Pa., 1958.

home-interview survey. The application of this index of transit usage was based on the assumption that a three dimensional plot of the characteristics of variation in transit usage would maintain the same form and shape from one city to another. The number of person work trips made by auto for each zone was then obtained by subtracting these transit work trips from the total person work trips for each zone. To adjust for car occupancy and to arrive at auto-driver work trips, the number of persons per car was applied to the total number of automobile person work trips previously developed for each zone. The relationship between car ownership and car occupancy is shown in table 13.

For each of the 10 external stations in Sioux Falls, the number of automobile work trips produced by each station was estimated as a percentage of the adjusted total number of trips for all purposes recorded at all stations during a standard external cordon survey. The adjusted total number of trips for all stations was obtained by deducting the number of through trips from the total number of external station trips and analyzing the remaining number of trips. The adjusted total number of external station trips consisted of auto and taxi trips between the external stations and the zones. The percentage of automobile work trips produced by the 10 external stations was determined to be 20 percent of this adjusted external station volume.

To determine the accuracy of these procedures, the number of auto-driver work trip productions estimated for each zone was

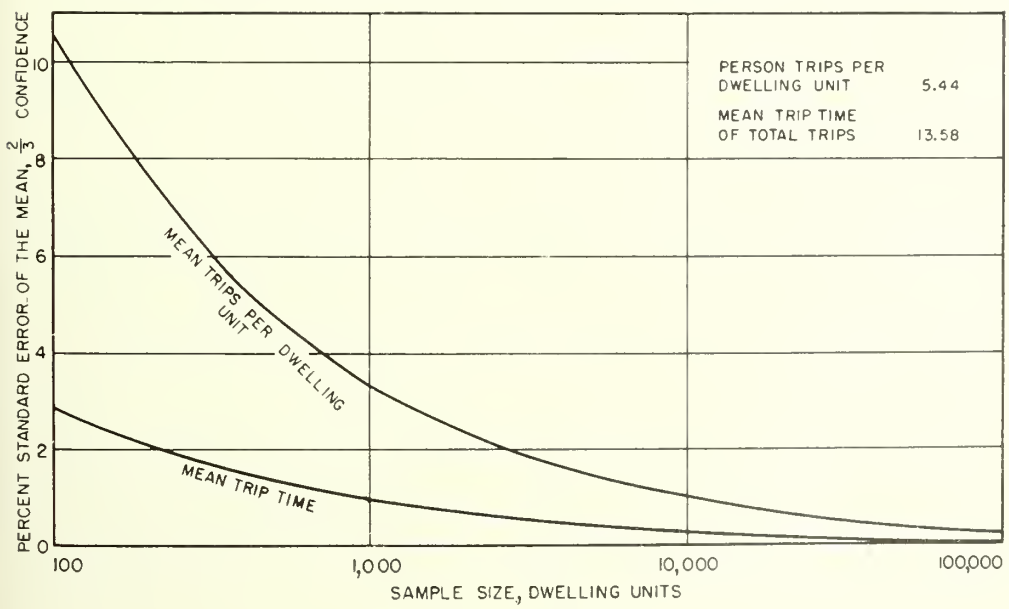

Figure 15.—Percent standard error versus sample size in dwelling units (log scale) for mean trip time and trips per dwelling unit, Pittsburgh, Pa., 1958.

Table 13.—Relationship between persons per car and car ownership for total work trips [1]

Cars per 1,000 persons	Persons per car
500	1.20
450	1.23
400	1.27
350	1.30
300	1.33
250	1.40
200	1.46
150	1.52
100	1.65

[1] *From Use of Mathematical Models in Estimating Travel*, by Alan M. Voorhees, Journal of the Highway Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HW4, December 1959, pp. 131–132.

compared with those from the 1956 O–D survey. These comparisons were also analyzed, using the RMS error criteria, and the analysis showed very close agreement between the actual and the estimated figures. The limits of 1 RMS error are shown as dashed lines in figure 16.

Zonal trip attractions

Zonal work trip attractions for each of the 74 internal zones were developed from zonal employment information. Information on the number of people employed in each zone and other statistics had been collected in a special survey by the Sioux Falls Chamber of Commerce. An analysis of these data showed that each employee in Sioux Falls was responsible for about 0.83 one-way person work trips per day. To obtain an estimate of the total number of person work trips attracted to each zone, zonal employment figures were multiplied by 0.83. Corrections were then made for transit usage and car occupancy to arrive at the number of auto-driver work trip attractions—information in table 12 and the tabulation for work trip productions were used. A control figure for the number of work trips to the CBD was also applied. Essentially, the estimated number of auto-driver work trip attractions to the CBD were factored to agree with the number from the small sample and the external survey. All auto-driver work trip attractions to non-CBD zones were factored in a similar manner so that the total number of auto-driver work trip attractions remained the same.

For each of the 10 external stations, the number of auto-driver work trip attractions was determined in the same manner as the number of external station auto-driver work trip productions. The percentage of total station auto-driver trips (minus through trips) that were attracted by the external stations was determined to be 6.0 percent.

To assure the accuracy of these procedures, the total number of auto-driver work trip attractions estimated for each zone was compared with the attractions from the 1956 O–D survey (figure 17). These comparisons were analyzed in the same manner as the work trip productions, and the actual and the estimated figures agreed closely.

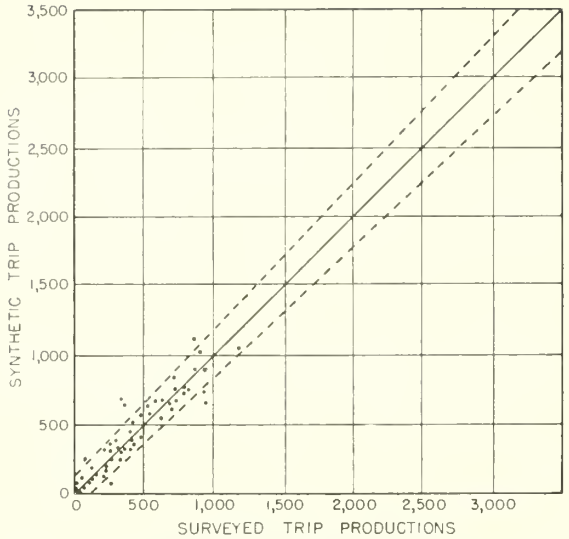

Figure 16.—Synthetic versus surveyed auto-driver home based work trip productions, Sioux Falls, S. Dak., 1956.

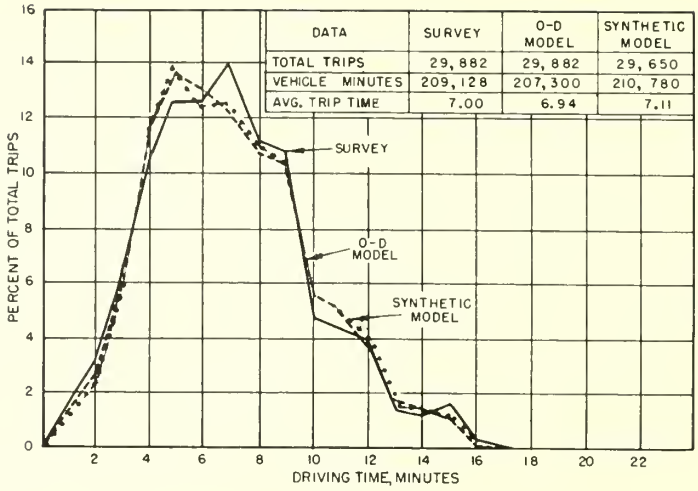
DATA | SURVEY | O-D MODEL | SYNTHETIC MODEL
TOTAL TRIPS | 29,882 | 29,882 | 29,650
VEHICLE MINUTES | 209,128 | 207,300 | 210,780
AVG. TRIP TIME | 7.00 | 6.94 | 7.11
SURVEY
O-D MODEL
SYNTHETIC MODEL
PERCENT OF TOTAL TRIPS
DRIVING TIME, MINUTES

Figure 18.—Trip length frequency distribution, home based auto-driver work trips, Sioux Falls, S. Dak., 1956.

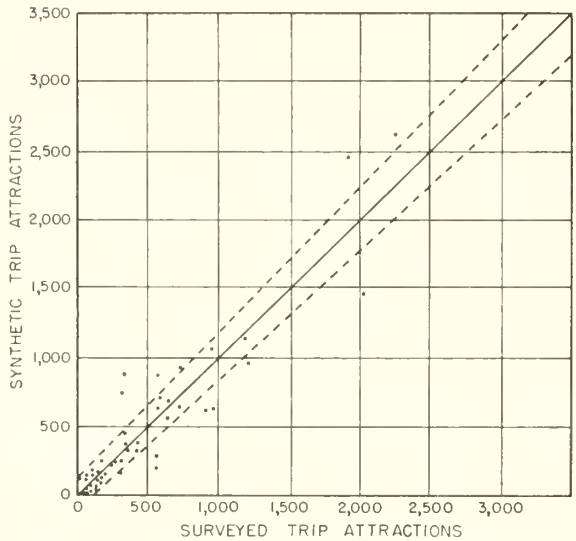

Figure 17.—Synthetic versus surveyed auto-driver home based work trip attractions, Sioux Falls, S. Dak., 1956.

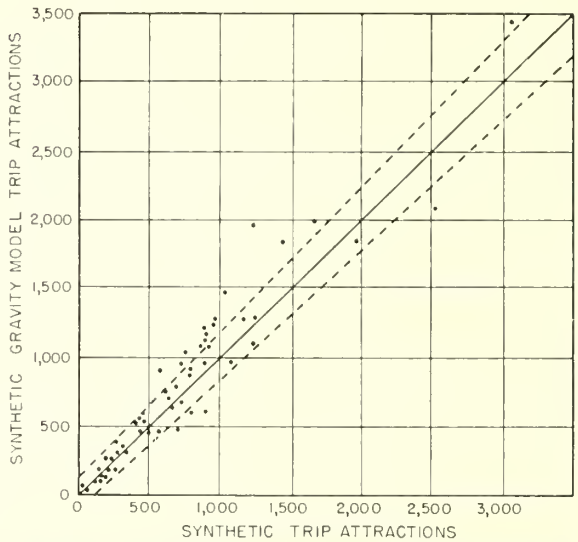

Figure 19.—Synthetic versus synthetic gravity model home based auto-driver nonwork trip deductions, Sioux Falls, S. Dak., 1956.

Home Based Auto-Driver Nonwork Trips

Zonal trip productions

Zonal nonwork trip productions in the 74 internal zones were derived from zonal data on car ownership, which were obtained from the O-D survey. The figure of 2.84 home based auto-driver nonwork trips per car (table 11) was applied to the number of cars owned by the residents in each of the internal zones to determine trip production figures for home based auto-driver nonwork trips. For the 10 external stations, the number of nonwork trip productions—30 percent of the total station volume—was obtained in the same manner as described for those in the external station auto-driver work trip productions.

To test the accuracy of these procedures, the total auto-driver nonwork trip production estimates for each zone were compared with the production from the 1956 comprehensive O-D survey and the actual and the estimated volumes agreed very closely.

Zonal trip attractions

Zonal nonwork trip attractions for the 74 internal zones were derived from zonal data on population and retail sales. The total number of internal auto-driver nonwork trip attractions divided into the total population figure for the area produced the population figure per attraction for this purpose. Repeating this process for the total retail sales in the area produced the unit of sales per attraction. Dividing the larger of these rates (population) by the smaller (retail sales) showed that 1.69 units of retail sales were required to attract each nonwork trip, although it took 1.00 unit of population to attract each nonwork trip. Using this technique a weighting factor

$$(\text{population} + 1.69 \times \text{retail sales})$$

was established as an indicator of the number of auto-driver nonwork trip attractions in each zone. The total number of attractions for this purpose was prorated to the zones by use of this weighting factor. The number of nonwork trip attractions also was factored to ensure that the number of CBD attractions was equal to those in the small sample survey data. The non-CBD attractions were adjusted accordingly to keep the total number of attractions the same as in the small sample.

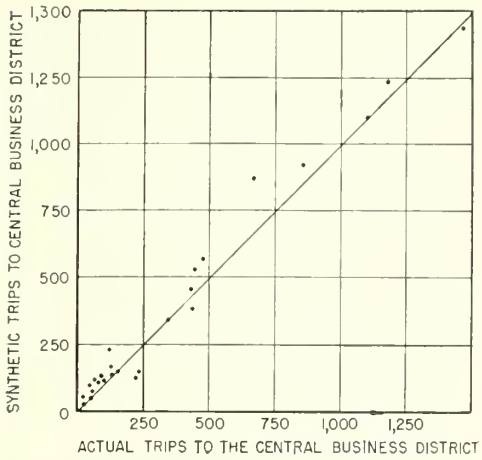

Figure 20.—Actual versus synthetic model nonhome based vehicular trips to CBD, Sioux Falls, S. Dak., 1956.

For the external stations, the number of auto-driver nonwork trip attractions was obtained in the same manner as described for the external station auto-driver work trip productions. Nonwork trips were 20 percent of the total station auto-driver trips (excluding the through trips).

To test the accuracy of these procedures, the total auto-driver nonwork trip attractions estimated for each zone were compared with the attractions from the 1956 O–D survey. The actual and the estimated volumes agreed reasonably well.

Table 14.—Differences between O-D data and synthetic gravity model estimates for district-to-district auto-driver trips, for three purposes, by volume groups [1]

Trip volume	O–D survey trips		RMS error	
HOME BASED WORK TRIPS				
Group	*Mean*	*Frequency*	*Absolute*	*Percent* [2]
0–99	21	400	20	95.24
100–199	133	40	58	43.61
200–299	259	13	119	45.95
300–499	402	13	98	24.38
500–1,499	920	8	186	20.22
HOME BASED NONWORK TRIPS				
0–99	27	423	28	103.70
100–199	136	53	83	61.03
200–299	239	28	103	43.10
300–499	380	22	166	43.68
500–999	728	22	282	38.74
1,000–2,999	1,711	9	343	20.05
NONHOME BASED TRIPS				
0–99	25	473	24	96.00
100–199	144	62	82	56.94
200–299	241	30	122	50.62
300–499	385	33	157	40.78
500–999	773	9	289	37.39
1,000–3,999	1,311	8	457	34.86

[1] 1956 O–D survey data versus synthetic gravity model estimates—relative difference measured in terms of percent root-mean-square error.

[2] Percent RMS Error $=100\left(\frac{\sqrt{\frac{\Sigma(d)^2}{n}}}{\bar{x}}\right)$

Where,
d= difference between surveyed and estimated trips
n= number of district-to-district movements
$\bar{x}$= mean of surveyed trips.

Nonhome Based Auto-Driver Trips

Zonal trip production and attraction

In several studies it has been reported that auto-driver nonhome based trip production is associated with car ownership (*4, 14*). Also, from the definition of trip production and attraction for nonhome based trips, production and attraction values should be equal on a *zonal* basis as well as on a study area basis. As trip origins should closely agree with trip destinations on a zonal basis during the 24-hour day, trip productions must also agree closely with attractions. This information was used in determining zonal trip productions and attractions for nonhome based auto-driver trips. Zonal nonhome based trip productions and attractions for the 74 internal zones were derived from the zonal data on car ownership. The figure of 2.98 nonhome based trips per car (from table 11) was applied to the number of cars owned in each internal zone to determine nonhome based trip productions. For the 10 external stations, trip productions and attractions were obtained in the same manner as described for external station auto-driver work trip productions. Nonhome based productions and attractions were 18.5 and 19.0 percent, respectively.

To test the accuracy of this procedure, the total nonhome based trip productions and attractions estimated for each zone were compared with those from the 1956 O–D survey. The actual and estimated values indicated poor agreement. An examination of the internal nonhome based trip productions and attractions from other studies also showed similar poor agreement. This is the most serious weakness of the small sample procedures.

Trip Distribution Patterns

The previously described procedures provide zonal trip production and attraction values for each of the three trip purpose categories. However, before trip interchange patterns could be calculated by using the gravity model, some measure of spatial separation between the zones had to be developed. For this phase of the research, the same information on the minimum path driving times between zones, the intrazonal times, and the terminal times was used as developed in the first part of this research. In addition, some measure of the effect of the spatial separation on trip interchanges between zones (F) was also required. In this phase of the research, full use was made of the traveltime factors already developed for each trip purpose. This was done because of the similarities of the trip time length frequency curves between the total OD sample and the subsample of 599 household interviews.

After all the required parameters had been determined, the gravity model calculations were made to obtain a synthetic trip distribution pattern. This synthetic pattern was compared with the O–D survey data to determine its accuracy. Several tests were involved in the comparisons. First, the synthetic trip time length frequency distributions and average trip time lengths were compared with those from the O–D survey for each trip purpose category; they agreed closely. The results for work trips are shown in figure 18.

The number of trips attracted to each zone, as computed by the gravity model, were compared with those shown by the synthetic procedures, for each trip purpose. The results for nonwork trips, which had the largest scatter, are shown in figure 19. Another test was made of the number of synthetic trips crossing the Big Sioux River. The figures compiled for this test were compared with those from the O–D survey (table 4). Again, the differences were small.

Synthetic trips to the CBD, for each trip purpose, were also compared with the actual O–D movements. The results for auto-driver nonhome based trips are shown in figure 20. No geographical bias was present in the synthetic trip interchanges, and the discrepancies between the two sets of information were very small. Synthetic trip interchanges for total trips were assigned to the minimum path driving time network. The expanded number of trips from the full O–D sample were likewise assigned. These two sources of information were then compared (table 5) by analyzing the differences over a comprehensive series of screenline crossings, these are shown in figure 5.

Finally, a statistical comparison of the actual and the estimated number of trips was made for each trip purpose. The results of the comparisons are shown in table 14. Results were acceptable for all purposes. The accuracy of this synthetic model calibration was equivalent to the accuracy attained with the model calibrated with all the Sioux Falls O–D data (table 6). The accuracy of the synthetic calibration also compared favorably with the results from other studies (*12, 15, 16*) when the models were calibrated with the O–D data.

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NEW PUBLICATIONS

Highway Progress, 1964

Annual Report of the Bureau of Public Roads Fiscal Year 1964

A review of the accomplishments of the Bureau of Public Roads, U.S. Department of Commerce, during the fiscal year 1964, particularly those related to the Federal-aid highway program, is presented in the annual report, *Highway Progress, 1964*. The 107-page illustrated publication contains a descriptive account of the progress made during fiscal year 1964 on construction of the National System of Interstate and Defense Highways, and on the improvement of primary highways, secondary roads, and urban arterials under the regular Federal-aid program.

Also described in the annual report is the highway construction work undertaken directly by the Bureau of Public Roads in national forests and parks and on other Federal lands, as well as Public Roads activities in providing technical assistance to foreign countries to further their programs of highway development.

Reported on at length are the activities and accomplishments of Public Roads in management, highway planning and design, urban transportation planning, safety, and in the extensive and varied research and development program. Statistical information on the progress and activities of the Federal-aid program during the fiscal year 1964 is presented in 19 statistical tables included as an appendix to the report.

Highway Progress, 1964, may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for 35 cents a copy.

Highways and Economic and Social Changes

The results of more than 100 economic impact studies in which State highway departments, universities, and the Bureau of Public Roads have cooperated are analyzed in *Highways and Economic and Social Changes*, which was issued in November 1964 by the U.S. Bureau of Public Roads and is now available from the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402, for $1.25. Because only a limited number of the individual reports were printed, this publication provides a historical record of the research in these areas through 1961. Since 1961 additional economic studies have become available, and other research has been started on the impact of highway interchanges upon adjacent areas and related subjects. The appendix contains lists of the economic impact and interchange studies that have been completed since 1961 and those that are now in progress.

The attraction of activities to highway channels and the relationships of highway improvements to economic and social change are described in the factual materials gathered for the studies in this publication. The dramatic changes wrought by highways upon people, homes, businesses, and land are discussed. This publication will be useful as a source book of the economic and social effects of highways for transportation research and planning personnel, as well as those in the fields of community planning, land acquisition, and economic development.

U.S. GOVERNMENT PRINTING OFFICE:1965

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REPORTS TO CONGRESS

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

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Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

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Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

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OF COMMERCE,
WASHINGTON

Interstate Highway 95, north of Fredricksburg, Va.

Design of this recently completed part of I-95 retained the natural scenic beauty and aligned roadways so that median growth and terrain screen opposing headlight glare.

IN THIS ISSUE: *Headlight Glare and Median Width—Three Exploratory Studies*

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IN THIS ISSUE

Headlight Glare and Median Width—Three Exploratory Studies, *by L. D. Powers and David Solomon* 125

Moving Picture Technique for Highway Signing Studies—an Investigation of Its Applicability, *by R. D. Desrosiers* 143

Motor-Vehicle Travel Estimates—1963 United States; 1962 State-by-State, *by T. S. Dickerson, Jr., and I. E. Corvi* 148

Films Available from the Bureau of Public Roads 151

New Publications and Films 142

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Headlight Glare and Median Width—

Three Exploratory Studies

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by[1] **LAWRENCE D. POWERS,**
Highway Research Engineer, and
DAVID SOLOMON, Principal
Research Engineer,
Traffic Systems Division [2]

The effects that glare from headlights of an opposing vehicle in relation to lateral separations (median width) have on meeting drivers are discussed in this article. Different methods of approach to the problems related to headlight glare and its effect in relation to median width were used in each of three studies. The test methods for each succeeding study were designed to compensate for the limitations of the preceding study and/or studies.

The conditions of each study were, as follows:

In Study 1, both the opposing glare car and target were stationary; the test subject drove toward the target and indicated when he could detect it. In Study 2, both target and test subject were stationary and the glare car moved toward the test subject; locations of the glare car were determined at which the target was just visible to the test subject. In Study 3, a self-luminous target was used and both target and test subject were stationary as the glare car moved toward the subject; he continuously adjusted the brightness of the target and attempted to keep it barely detectable. Some limited measurements of discomfort caused by glare were made in Study 1, but this line of investigation was abandoned because of the variability in the results and the lack of an adequate definition of discomfort.

As expected, study results showed that the effects of glare decreased with increasing lateral separation of the glare car and the opposing vehicle. At any given lateral separation, the effects of the glare were present even when the glare car was at a considerable distance from the observer (3,000 feet or more); the rate of change of the effect with distance was small for a large part of this distance.

The authors have made recommendations on how to conduct such target detection studies in the future in order to obtain the most meaningful results. The entire visual problem of driving at night is discussed and possible areas for future investigation are suggested.

Introduction

THE METHODOLOGY and the results of three exploratory studies on the relation of the median width of a highway and the disabling effects of opposing headlight glare are published in this article. The three studies, in each of which different test methods were used, were made to gain insight concerning the factors producing headlight glare and the relationship of these factors to median width. Because insight was the main objective of these studies, only small amounts of data were collected and little reliance has been placed on the quantitative results.

The methods used in each succeeding study were developed to compensate for the limitations revealed in the analysis of the preceding one. These studies were all made in the spring of 1961, but the preliminary report was not published because of the recognized limitations in the data developed or because of the difficulty in interpreting the results. Readings in the psychological and physiological literature have provided a basis for an interpretation of the results. Information is presented here for those who may be contemplating research in the same area. Therefore, considerable detail has been included on field layouts, procedures, and qualifications of results. Discussions of possible pitfalls and factors that may have to be considered are also presented.

The results of these studies were based on small samples and few replications. They are more suggestive than conclusive and are of value primarily to show the types of results that may be expected and to illustrate effects of certain factors. The absolute values of the results hold only for the particular targets, subjects, surroundings (pavements, backgrounds, ambient illumination), and lamps tested. The relative values of the results such as the effect of lateral separation may also be dependent on these variables.

It had been proposed that the discomforting as well as the disabling effect of headlight glare be studied. The discomforting effect, however was elusive of definition and more so of measurement. An attempt to measure it was made in Study 1, but the results were too variable to be of any use. Thereafter the objective was limited to what was thought to be the more critical problem of disabling glare.

Data for Studies 1 and 2 were collected in one night each and for Study 3 on two successive nights. The data are not considered to have been biased because of serial changes in conditions. Although different ambient illumination conditions may have existed on the different nights, it is assumed that they were fairly constant for the duration of each night; furthermore, balanced experimental designs and random orders of exposure were employed for the different conditions such as lateral separation. When the data are subject to question for other reasons, qualifications for the results are given. The existence of possible differences between nights, as well as differences in some of the other variables, reduces the comparability of the results of the different studies.

Notes and Definitions

The test site, 2 miles long by 150 feet wide, was a new portland cement concrete runway at Dulles International Airport, prior to its being opened to air traffic. The site was 25 miles from downtown Washington and about

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[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., Jan. 1964.

[2] The authors express their appreciation to W. P. Walker, who suggested these studies; R. M. Michaels, for providing consultation and advice prior to and during the studies; R. N. Schwab, for reviewing the visual theory contained herein; the Federal Aviation Agency, for permission to use the runways at Dulles International Airport; and to the many people who participated as test drivers or field crew: W. P. Walker, D. Merchant, R. D. Desrosiers, R. E. Payne, E. L. Poland, R. Wells, J. P. Eicher, and P. Granum.

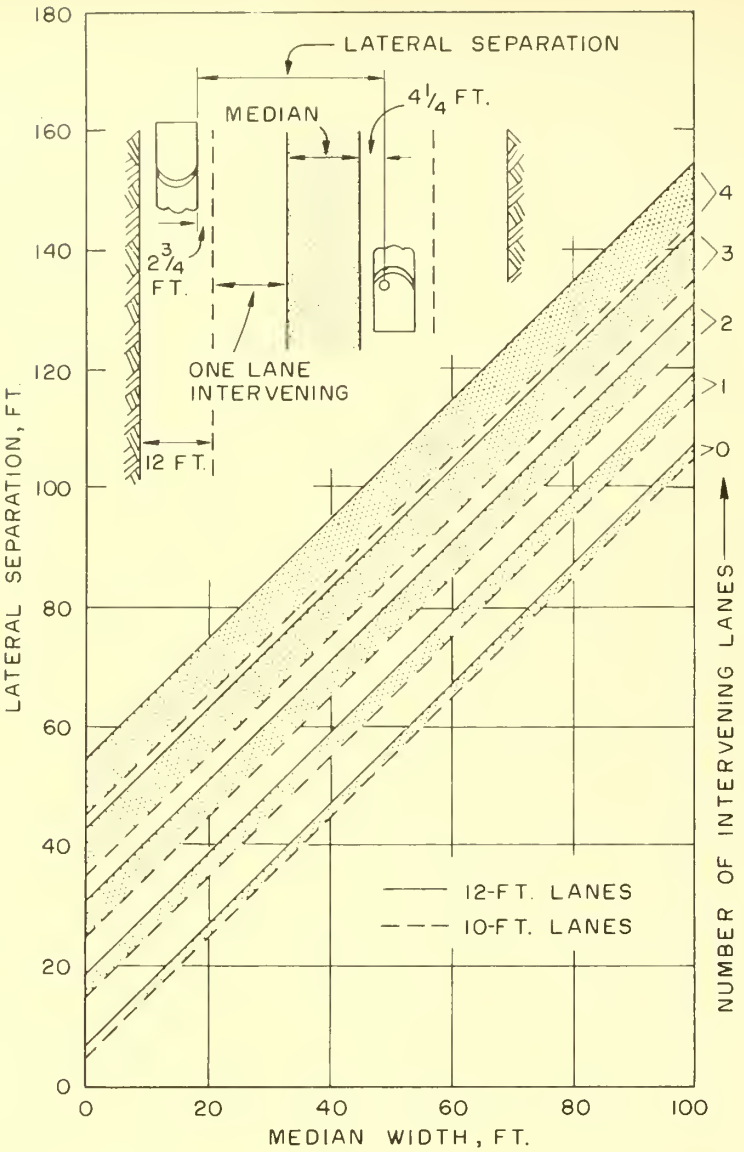

Figure 1.—Relation between lateral separation and median width for different combinations of vehicle position and lane width.

12 miles from the nearest sizable town. The area surrounding the runway was flat and grassy. Consequently, the ambient illumination was uniform and at a very low level. Few, if any, extraneous light sources were visible, depending on the direction in which the test subjects were facing for the different studies. When present, these lights were distant and constant during the collection of data. For the conditions of the studies, the effects of cross-slope of the runway were assumed to be negligible so that, except where noted, all studies were performed on portions of the runway considered to be a plane surface.

The tests simulated only the meeting of a single vehicle and a single opposing *glare* vehicle on a constant-grade, tangent section of highway, where, at any cross section, the pavements for both directions of travel were at the same elevation. The results do not necessarily hold for other geometries.

All vehicles employed in the studies had four headlamps mounted horizontally. Prior to each study, the aim of the headlamps of both the glare car and the car for the test subjects was adjusted according to Virginia State standards. All tests were conducted with the glare car headlights on high beam. The windshield of the car in which the test subjects were seated, the outsides of the headlamps of both the glare car and test subject's car, and the reflectors on the target car were maintained clean throughout the course of the studies.

Lateral separation

Instead of median width, the more definitive concept of *lateral separation* is used in this report. As employed here, *lateral separation* refers to the lateral distance between the test subject and the near side of the opposing glare vehicle. Lateral separation can be used to convert (figure 1) to different combinations of median width, vehicle position, and lane width. For the conversion it has been assumed that each of the two vehicles is 6.5 feet wide and centered in its lane; the test subject's eyes are assumed to be 1.5 feet from the left side of his vehicle. Following is an example of how figure 1 may be used, when conditions are: median width, 20 feet; four-lane divided highway; both vehicles in their respective right lane (two lanes intervening); lanes, 12 feet wide; then the resultant lateral separation is 51 feet.

The condition in which no opposing vehicle was present can be thought of as corresponding to an infinite lateral separation. For convenience, this condition (absence of opposing vehicle) is referred to as the *no-glare* condition although, technically, any light in the field of view will produce some glare, such as the area of pavement illuminated by the driver's own headlights.

Longitudinal separation

Although the tests were run at slow speeds and for long distances—2,000 feet or more—these distances were not so extreme as they may appear at first glance and actually were not long enough to develop all the needed data. On modern, divided highways, sight distances of several thousand feet to opposing headlights are fairly common. Two opposing vehicles, each traveling at 70 m.p.h., will have a relative velocity of about 200 feet per second. If the two vehicles are initially 2,000 feet apart, they will meet in 10 seconds.

Threshold

Psychophysical *absolute threshold*, according to Stevens (*1*),[3] is the level of a stimulus—one example is the brightness of an object—that marks the transition between response (detectable) and no response (nondetectable). This threshold level ordinarily fluctuates from moment to moment within a range separating those stimuli levels that definitely produce a response from those that definitely do not produce a response. A given stimulus will produce a response only part of the time. The probability of detection of a stimulus, therefore, will range from close to zero to almost 100 percent. Usually, the stimulus level that produces a response 50 percent of the time is called threshold (50 percent threshold). For identical conditions, the average thresholds for different subjects may be different. The term *threshold* has been used throughout this article, but it is used in a much less restrictive manner and refers, generally, to when the test subject reported that he detected or lost sight of the target. Obviously, the conditions of exposure of the stimulus—time of exposure of the target, subject's adaptation levels, and criterion for confidence of detection—could not be controlled so precisely as in a less realistic laboratory setup, and the individual threshold measurements reported herein could have differed considerably from the 50-percent detection level.

[3] References indicated by italic numbers in parentheses are listed on p. 141.

STUDY 1

The field layout [4] for Study 1 is shown in figure 2. The headlights of both the glare car and the test subjects' car were on high beam. The glare car and the target car were both stationary, the rear of the target car being 100 feet beyond the headlights of the glare car. Of the five test subjects, three in their 20's and one 54 years of age reported normal or corrected-to-normal visual acuity; one, age 33, reported poor acuity.

Each test subject, using a pavement joint as a guide, drove toward the rear of the unlighted target car at a speed of 20 miles per hour. His instructions were to call out to a recorder seated in the car when he felt the glare to be discomforting and again when he could discern any part of the target car (the part of the target car first detected in every run was one or more of the six, rear red reflectors). Distance markers, 40 feet apart, lined the edge of the runway. The recorder would note the position of the car relative to the nearest distance marker; from this information the longitudinal distance could be estimated to within 20 feet. The course markers carried reflectorized letters that were turned away from the view of the test driver. The recorder viewed them by shining a flashlight out the side of the car.

Each test subject made three runs at each of four different lateral separations of 7, 32, 57, and 82 feet, and also for the no-glare situation, which corresponded to an infinite separation. Runs were made in random order, although each subject made all his runs consecutively. For each different lateral separation the target car was positioned so that it was directly ahead of the test subject's car.

The method used in Study 1 is similar to that used by the Idaho Department of Highways in a 1957 glare test (*2*).

The glare car was not moved at all during Study 1, as all data were collected in one night. Therefore, the orientation of the headlight candlepower distribution was constant for all runs. However, it is not known whether the axis of the glare car, and consequently of the headlights, was exactly parallel to the line of travel of the test subject.

Discomfort Results

Study 1 is the only study of the three for which data relative to both disability and discomfort glare were obtained. The results of the discomfort test showed that the discomfort measurements were highly subjective and variable.

For the different lateral separations, data in figure 3 show the distances from the glare vehicle at which subjects stated they experienced discomfort from the opposing headlights. The grid portion of the figure can be viewed as a plan of the test site. Each subject is identified by a different letter, A through E. The points are plotted at the distances for each run at which the test subject reported discomfort; when the subject reported no discomfort during a run, a point is shown to the left of the zero line. For two runs at each of the two narrowest separations, subject E reported discomfort at the beginning of the run before he arrived alongside the beginning of the series of distance markers (2,520 feet). Therefore, these distances are unknown, but were more than 2,520 feet and have been plotted arbitrarily at 2,700 feet.

A curve has been drawn for each subject. These curves are approximate because of the extreme scatter of the data. Had each curve been drawn to pass through the mean discomfort distance for each lateral separation, some of them would have been S-shaped. As had been expected, the distances at which the onset of discomfort was reported generally increased as lateral separation was decreased; or, to put it simply, discomfort was experienced sooner when the lateral separation was narrow. When 1,000 feet from the glare source, for example, subject A stated that he experienced discomfort only when the lateral separation was 7 feet. Discomfort was reported by the others, as follows: subject B, when the separation was 32 feet or less; subject C, when the separation was 57 feet or less; and subjects D and E, even when the lateral separation was 82 feet.

For any given run, as the distance between the subject and the glare car decreased and the opposing headlights became farther from his line of sight and decreased in brightness, obviously points had to be reached at which discomfort began to diminish and at which it ended, even if one or both of these points were where the driver passed the glare car and the headlights were no longer visible. Therefore, the discomfort threshold curves must bend over at small longitudinal distances. Data were not collected for these points on the discomfort curves, but for illustrative purposes they are shown as the broken-line parts of the curves for subjects A, B, and C.

Part of the differences in results may have been the result of the subject having had two tasks to perform: to report discomfort and to report detection of the target. As detection of the target was the more emphasized task, subjects may have failed to concentrate or report on discomfort if it occurred at the same time that the target became detectable. But the differences in results among subjects undoubtedly were caused both by differences in sensitivity to glare and individual definitions of discomfort. However, it was hard to believe that the large scatter for any individual could be attributed to the small sample size alone, and it became apparent that discomfort glare criteria were going to be difficult to define. Subsequent research was therefore limited to the study of only disability glare.

Target Detection

Data in figure 4 show how target detection distances change with lateral separation. The part of the target car detected first was always one or more of the six, rear red reflectors. Each plotted point represents one run by one subject at the lateral separation indicated. The data for each subject are shown by a different symbol. The data seemed to fall into two distinct groups, apparently according to the nighttime visual ability of the subjects. Consequently, they have been so grouped. The solid curve is for the three subjects in their 20's, who had reported normal acuity. The lower dashed curve is for the other subjects: the one 33 years of age who had reported poor acuity and the one 54 years of age who had reported normal acuity. For purposes of identification, the group of three, who were judged to have normal visual ability, have been given the designation N; the other two subjects, whose performance was judged to be somewhat lower, have been designated L. Within each group the differences between subjects appear small and curves drawn for individual subjects would approximate the curve for that particular group. The alternate dashed and dotted curve shows an average for all five subjects (fig. 4).

As had been expected, the distances at which the target was detected increased as lateral separation increased. Average detection distances appeared to approach those of the no-glare condition at lateral separations of approximately 80 feet. However, the curves for the two sets of subjects are somewhat different in that the curve for the two subjects (L) approaches a maximum at a slightly narrower lateral separation compared to the curve for the three subjects (N). The apparent anomaly for the two subjects (L) of greater detection distance at the 80-foot lateral separation than for the no-glare situation can be attributed to the small sample size. Otherwise a new theory would be called for. Data from the Idaho study, in which a different target was employed, have also been plotted. The median widths given in the Idaho report have been adjusted to the same lateral separation assumption used for Study 1. Data for wider separations or for no-glare situations were not included in the Idaho study, so the shapes of the curves for the different targets cannot be compared.

Visibility and Discomfort Compared

The performances of the subjects in the visibility test of Study 1 were compared with the discomfort test to see whether there was any correlation between the two measurements. The two subjects (L), who showed somewhat poorer visual performance, are shown as B and E in figure 3 for the discomfort test. Subject E, age 54, reported discomfort at all lateral separations; whereas B, age 33, who had reported poor acuity, reported discomfort for only the two narrowest separations. The discomfort results for the other three subjects, A, C, and D, were divergent,

[4] As employed in this article, *layout* refers to the plan of the site for each study; *setup* refers to the relative position of glare car and target car.

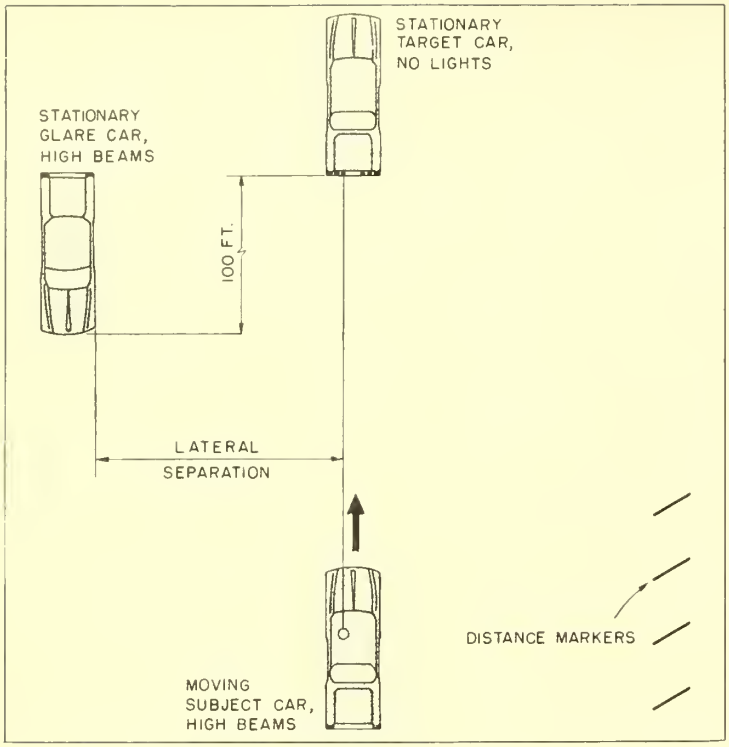

Figure 2.—Field layout for Study 1.

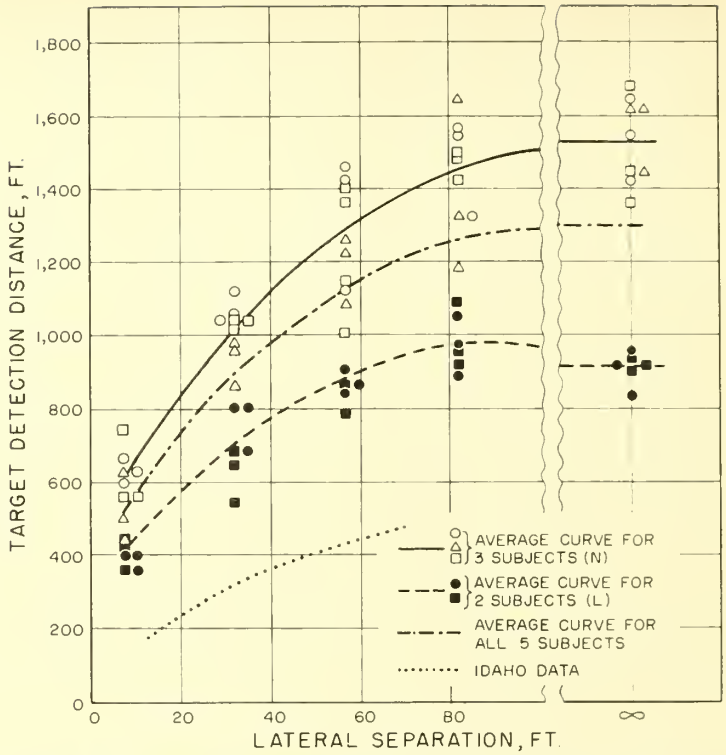

Figure 4.—Target detection distance by lateral separation for five subjects in Study 1, target 100 feet beyond glare car. (Idaho data are for a different target.)

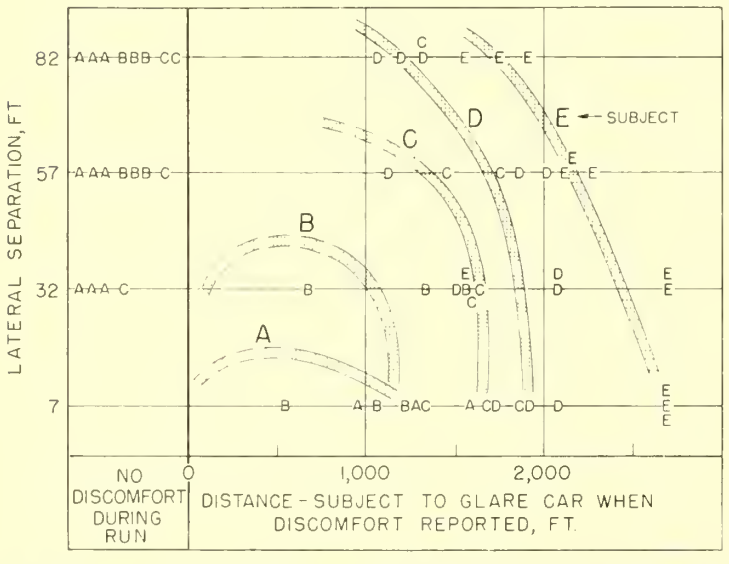

Figure 3.—Effect of position of opposing high beams on discomfort reported by five subjects. Each lettered point is plotted at distance at which subject reported onset of discomfort for each run at given lateral separation. Presence of discomfort was assumed for combinations of lateral and longitudinal separation from glare car that lie under the curve.

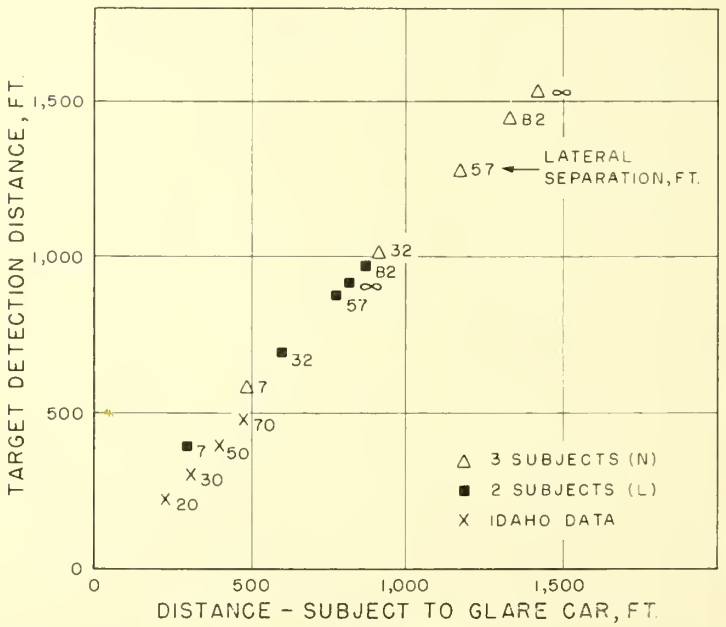

Figure 5.—Relation between target detection distance and distance of glare car for different lateral separations for the method of Study 1. (All data fall on a line, which has a slope of unity and an intercept of 100 feet.)

whereas their performances in the visibility test were nearly identical. On the basis of these limited data, therefore, there is no evidence of a correlation between visual sensitivity and discomfort sensitivity to glare.

Limitations of Study Procedure

The differences in the target detection distance curves for the two groups of subjects may be related to the fact that automobile headlamps send out focused beams with different light intensities at different angles from the axis of the lamp. This may explain why the curve for the two subjects (L) approaches the no-glare value at smaller lateral

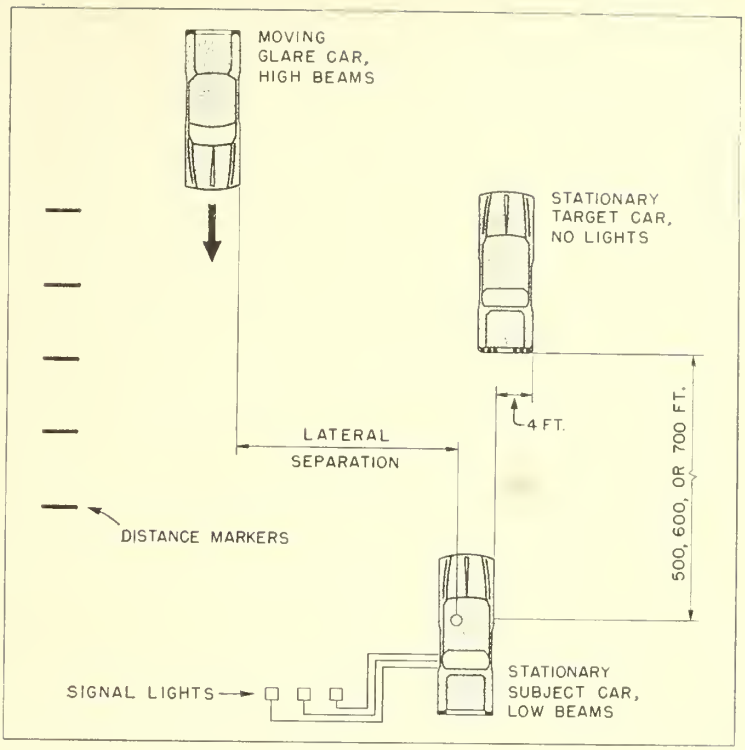

Figure 6.—Field layout for Study 2.

separations than does the curve for the three subjects (N). Because of poorer visual ability, even without glare, the former group had to be closer to the target to detect it under any condition. Being closer to the target, they were also closer to the glare car but were subjected to lower intensities of opposing glare. The three subjects (N) were able to detect the target at longer distances for the no-glare situation. At these longer distances and the same lateral separation, they were subjected to higher levels of glare and the extent of the reduction in target detection distance was greater. This same interaction between level of glare and detection distance may also produce different relationships between lateral separation and detection distance for different targets because the observer will be in a different orientation to the opposing beam for each target at the time it becomes detectable.

Reevaluation of the study procedures for Study 1 showed conditions that limited the applicability of the results obtained for disability glare. Among these were the dependence of the results upon the particular geometry, headlight configurations, test subjects, and target used in this experiment. A major limitation of the data, however, was that the relative positions of target and glare car were fixed, as shown in figure 2. As the subject approached both the glare car and the target, his position with respect to the opposing headlight beam pattern changed simultaneously with the change in apparent size of the target and the illumination on it from his own headlights. Regardless of when the subject could detect the target, the detection distance was always equal to the glare car distance plus 100 feet. This is further illustrated in figure 5, in which a comparison is made of detection distances with distance to the glare car. All the data of Study 1 fall on a line; the equation is:

$$D_t = D_g + 100 \quad (1)$$

Where,

D_t = target detection distance,
D_g = distance, driver to glare car.

Similarly, the Idaho data fall on a line:

$$D_t = D_g \quad (2)$$

because the target was adjacent to the glare car. When this study methodology is used, detection distances of the target cannot be determined for other relative positions of glare car and target car. For example, if the subject were 1,000 feet from the glare car and the lateral separation was 7 feet, at what distance would he have been able to see the target. In attempting to describe the relationship between the effects of headlight glare and lateral separation, it is necessary to know whether the detection distances derived were the most critical distances. The question remains, in regard to the results of Study 1, were these the minimum detection distances for each lateral separation?

The lack of applicability of the data that was caused by the study method was the result of the lack of independence of the two variables: distance to target and distance to glare car. Therefore, this study method was abandoned in favor of one in which only one distance at a time was changed.

STUDY 2

Essentially, the objective of Study 2 was to determine the positions of the glare car for each of the specified target detection distances. The field layout for Study 2 is shown in figure 6. In this study the subject and target cars were stationary and the glare car, which had high beams on, was moving. The subject car's headlights were on low beam. This is considered to be the worst condition for a two-car meeting situation.

The three test subjects reported visual acuities ranging from poor to good. Each subject was provided a pushbutton that he could use to operate a signal light, a different color for each subject. The signal lights, consisting of 1-foot squares of red, yellow, or green translucent plexiglass illuminated from behind by incandescent bulbs, were situated out of the view of the subjects and sufficiently far apart to be distinguishable at distances up to 3,000 feet by the experimenters.

The three subjects, seated in the subject car (two in front, one in rear), viewed the rear of the target car as the glare car moved toward them. As long as the reflectors of the target car were visible to him, each subject kept his particular signal light on. Subjects were instructed not to communicate with each other during a run.

The same distance markers that had been used in Study 1 were placed at intervals of 100 feet along the side of the test site so they were facing the glare car. The glare car contained three men: a driver, an observer, and a recorder. The driver's func-

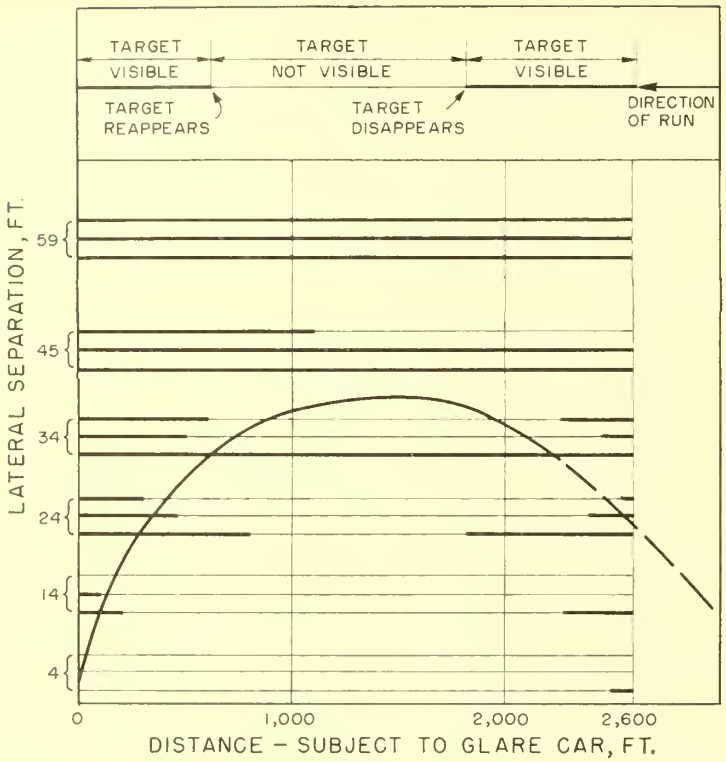

Figure 7.—Target visibility data and threshold curve for one subject, Study 2. (The target, 700 feet from the subject, was not visible when glare car was at combinations of lateral and longitudinal separations below curve.)

tion was to stay on course, maintain a speed of 25 m.p.h., and call off the distance markers as they were passed. The observer would call out the status of the signal lights, on or off, and the recorder would enter this on the data sheet for the appropriate marker designation. Distances were estimated to the nearest 50 feet. The farthest marker was at a distance of 2,600 feet from the subject.

Three runs were made at each of six different lateral separations (4, 14, 24, 34, 45, and 59 feet) and for each of three different distances of the target car: 500, 600, and 700 feet. The irregular figures were used for lateral separations because of the convenience of using the pavement joints as tracking guides. Separations of 4 and 14 feet were intended to bracket the zero median test situation, which for lanes 12 feet wide would provide a lateral separation of 7 feet. The target car was offset 4 feet laterally to the right of the subject car, primarily to provide clearance for and prevent displacement by the glare car during runs at the narrowest separation.

The order of presentation of the different target distances was arranged in sets; a set consisted of one each of the three target distances, 500, 600, and 700 feet. For each target distance within a set of three, one run was made at each lateral separation, in random order. The order of target distances within a set was random; three sets were run.

The distance at which each of the three drivers could see the reflectors on the target car without the presence of the glare vehicle was recorded once, respectively.

Development of Data

The following detailed description and discussion of the development of the data from raw to final form will be of value because many factors involved in visibility testing are pointed out.

Figure 7 shows the positions of the glare car at which the target was visible at 700 feet to one of the subjects. Data for three replications for each lateral separation are shown. The figure can be viewed as a plan of the test site. The glare car ran from right to left; the measurements started at a longitudinal distance of 2,600 feet from the test driver. At some point the test subject could no longer see the target: visibility fell below threshold. As the glare car continued its run, a point was reached at which the subject could again see the target. An average threshold curve has been roughed in. Points below the curve represent positions of the glare car—combinations of longitudinal and lateral separation—at which the target was not visible to the subject at a distance of 700 feet; that is, the target detection distance was less than 700 feet.

When lateral separations were narrow, the target was often below threshold for the subject before distance measurements could be taken on the glare car. Consequently, the far end of the threshold curve (fig. 7) has been drawn with a broken line to indicate the extreme uncertainty of its location. The curve has been extrapolated beyond 2,600 feet in order to illustrate the method. Had the glare car measurements been started at a longer distance, there would have been some point at which the target would have been above threshold.

Also for narrow lateral separations, the target did not rise above threshold until after the glare car had passed the subject. This could have been caused by: the timelag in the subject's readapting to the no-glare condition, reaction timelags in actuating the signal buttons, and reaction timelags in the observing-recording process. All of the longitudinal distance data are probably too small because of timelags in the signaling-observing-recording process. To simplify the illustrations, the threshold curves of figures 7 and 8 have been drawn as though the target were always visible after the glare car had passed the subject.

At the wider separations, the target was sometimes above threshold during the entire run of the glare car; that is, the target detection distance was always more than 700 feet. This was true for one of the runs when the separation was 34 feet, for two runs when the separation was 45 feet, and for all three runs when the separation was 59 feet (fig. 7). As threshold is usually defined as a 50-percent probability of detection, the threshold curve has been drawn to level out at the lateral separation where the target was detectable 50 percent of the time, or 1.5 times out of three runs. Therefore, the curve has been drawn to level out in the vicinity of the 1,500-foot longitudinal distance, where a minimum point would appear to be, and midway between the 34- and 45-foot separations where the target was detectable for the entire run, once and twice out of three runs, respectively. Many more replications would be needed to determine the exact location of the average threshold curve. For purposes of illustrating the method, it was assumed for this analysis that 50-percent detection threshold data were being obtained. However, it is doubtful that the points where the target appeared or disappeared represent 50-percent detection thresholds.

Figure 8 shows average threshold curves for the combined data of all three subjects for each of the three target distances. The parts of the curves shown with broken lines are uncertain for the same reasons that the broken lines in figure 7 are uncertain. The curves can be thought of as representing contours of equal disability caused by glare, as the target detection distance was constant for all positions of the glare car that lie along each curve. The target was not visible at the distance shown for each curve for combinations of lateral separation and longitudinal distance of the glare car that lie under the curve. In figure 8 the shapes of the curves as drawn are deceptive because of the difference in the horizontal and vertical scales; the curves actually are very long and flat, indicating that the effect of the opposing glare on target detection was fairly constant over a long distance from the glare car.

The target detection distance can be shown in another way. By noting the distances of

the glare car at which a horizontal line, representing a chosen value of lateral separation, intersects the threshold curves of figure 8, the relation between target detection distance and distance of the glare car for that particular lateral separation can be obtained. Points of intersection so obtained have been plotted in figure 9 for arbitrarily chosen lateral separations of 7, 17, 27, and 37 feet. At separations wider than 37 feet, the target detection distance was always more than 700 feet and, as target distances of more than 700 feet were not tested, no points could be obtained for these wider separations. Curves showing changes in target detection distances by position of the glare car for the chosen lateral separations have been drawn through the points. Again, the uncertain parts are shown by broken lines. The average no-glare detection distance of 940 feet for the three subjects was derived from only one measurement per subject and is, therefore, shown by a broken horizontal line in figure 9.

Study 2 Results and Discussion

The curves (fig. 9) illustrate that target detection distance was relatively high at long distances from the glare car. This is reasonable, as the illumination reaching the subjects' eyes from the opposing headlamps was small. Target detection distance decreased gradually as the glare car got closer to the subject, reached a minimum, and then rose again as the glare car, continuing its run, moved away from the center of the visual field and the less intense part of the opposing headlight beam was directed at the eyes of the subject. As the lateral separation increased, the minimum target detection distance that was reached was higher and appears to have occurred at farther distance from the glare car.

The extent of the curves and the low rate of decline in target detection distance in relation to glare car distance show that the effect of the opposing glare on target detection distance extends for a considerable distance from the opposing glare car and is fairly constant over most of that distance. This was previously pointed out for the threshold curves of figure 8, from which the curves for figure 9 were derived. Also, as previously stated, at lateral separations of more than 37 feet, the average target detection distance was more than 700 feet. Similarly, the curve of target detection for the 7-foot lateral separation dips below 500 feet to an unknown point. Therefore, to have obtained data from which to draw the curves for lateral separations of more than 37 feet and to have obtained the minimum target detection distance for the 7-foot lateral separation, target distances of more than 700 feet and less than 500 feet would have had to be tested. The minimum points for the curves fall at points for target detection distance at which no data were obtained. For a more accurate determination of the minimum target detection distance and its location for each lateral separation, intermediate target distances would have had to be tested.

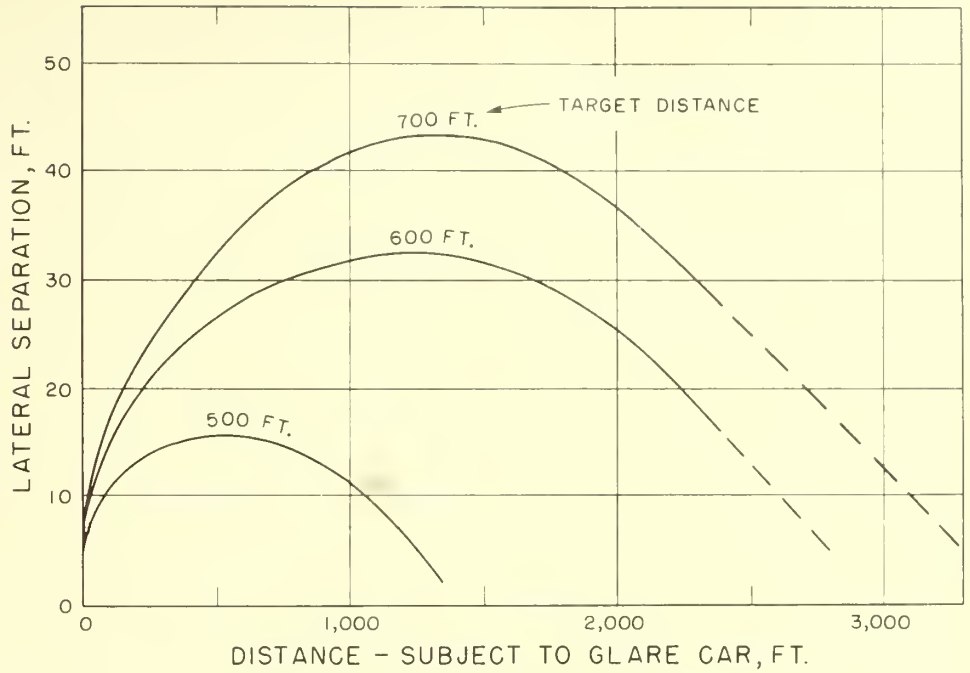

Figure 8.—Average threshold curves for three subjects of Study 2 for each of three target distances. Target was not visible at distance indicated on each curve when glare car was at combinations of lateral and longitudinal separations below curve.

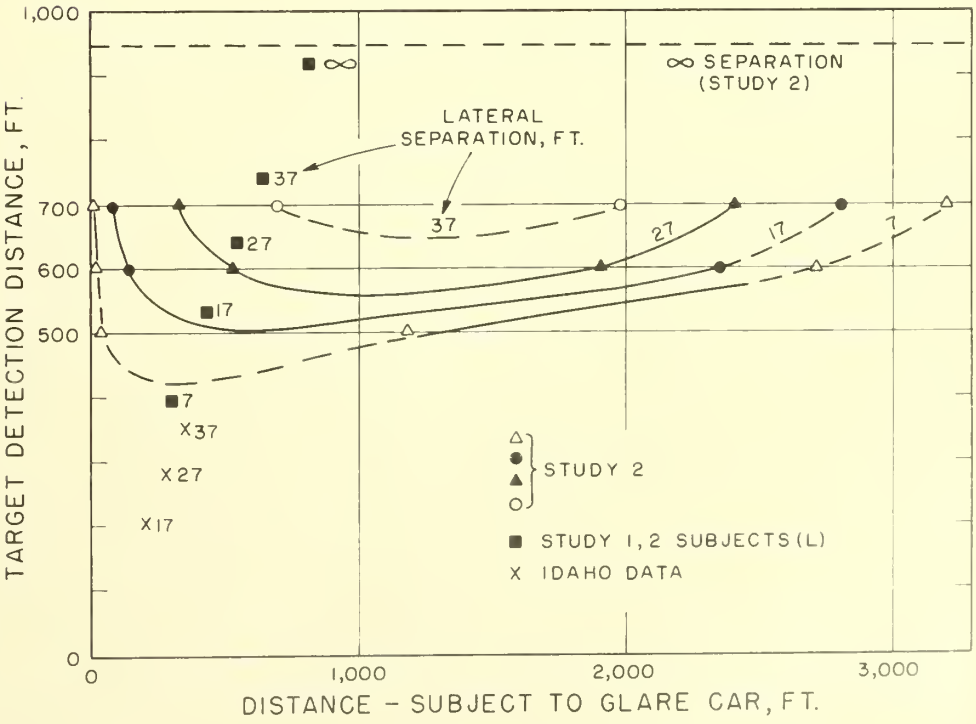

Figure 9.—Target detection distance in Study 2 by glare car distance for different lateral separations. (Partial data of Study 1 and Idaho study shown for comparison.)

The slope of the curves for glare car distances beyond 2,000 feet is uncertain because these parts of the curves were based on extrapolations of the threshold curves. Another cause for suspicion about the shape of the curves in this area is inherent in the study method. Because the target was at a fixed distance from the subject, the threshold point of detection for long distances of the glare car was determined when the target, initially visible, became invisible. The subject was therefore able to fixate on the target and could maintain detection of it for a longer time, as the glare car approached, than if the target had been initially invisible and then became visible. It is possible that target detection distances for the long distances of the glare car should be less than shown. The shapes of the curves obtained in similar studies by the Road Research Laboratory in England (*3, 4, 5*) do not have as great a slope as the curves of figure 9 at the long distances of the glare car.[5]

The roundabout method of developing the data just demonstrated, which involved drawing the threshold curves (fig. 8) and intersecting them with chosen lines of lateral separation in order to obtain points with which to plot the target detection distance curves (fig. 9), was used to illustrate some important concepts such as the variability in threshold measurements.

The average distance of the glare car for threshold detection of the target could have been plotted for each target and lateral separation directly on figure 9. It would not have been possible, however, to have interpolated for lateral separations other than those at which data were actually obtained.

Disadvantages

Although the method described in the preceding paragraph is direct, the problem remains of what to do where some, but not all, runs at a particular lateral separation do not provide threshold points because the target was detectable for the entire run. This points up a fundamental disadvantage of the method used in Study 2. Because the use of this method yields distances from the glare car at which target detection distance is constant, the data essentially fall on lines of constant target detection distance, as in figure 9. The curves of target detection distance by glare car distance that are derived from these data are long, flat curves, indicating the glare levels (disability contours) that cause particular threshold detection distances, which are long in extent and roughly parallel to the line of travel. If these disability glare levels can be visualized as long and thin, cigar-shaped contours emanating from the glare car, it can be seen that their intersection with the subject's line of travel is at a very small angle. This small angle of intersection combined with the variability inherent in threshold measurements produces such a high degree of uncertainty in the location of the actual threshold points that the number of repetitions required for accuracy become impracticable.

Study Methods Compared

In order to compare the attributes of the two study methods discussed so far, the data of the two subjects (L) who showed poor performance in Study 1 are shown on figure 9. Although these data of Study 1 closely approximate those of Study 2, the data of the two studies are not exactly comparable because of the differences in subjects, beam configurations, and methods of testing. Therefore, the fairly close apparent agreement may be mere coincidence. The data for the other three drivers (N) who took part in Study 1 were omitted from figure 9 to conserve space. For the same lateral separations, these subjects were able to detect the target at significantly longer distances; this illustrates the wide differences that may exist between different groups of drivers.

Because a fixed relationship between the glare car and the target was used in Study 1, all the data fell on a straight line (fig. 5). In figure 9, this line from figure 5 intersects each lateral separation curve at only one point, and these points are not necessarily the minimum detection distances for each lateral separation. This illustrates the disadvantages of using only one fixed relationship between glare car and target.

The retroreflectors used as the target in these studies provided a relatively high contrast task. These reflectors have a reflection factor in the order of 1,000, whereas all objects that reflect diffusely have reflection factors smaller than unity. The data of the Idaho study, for a target with a reflection factor smaller than unity, are also shown in figure 9 to illustrate the differences in the magnitude of the detection distance that will be obtained by the use of different targets. Because different targets will be detected at different distances under any condition, they will be detected at different distances from the glare car when it is present. The subject will be exposed to different levels of glare at the point of detection because he will be oriented differently to the opposing headlight beam pattern. Therefore, the curves for other targets might have different shapes than those that were derived from Study 2.

STUDIES 1 AND 2 SUMMARIZED

Because target detection distance was used as the measure of visibility in Studies 1 and 2 and not in 3, the results of the first two studies are summarized here. Also, methods for conducting detection distance studies are evaluated, a recommended method is described, and the use of detection distance as a parameter of visibility is discussed.

One surprising result of the first two studies was the extent, in terms of distances between the subject and the opposing vehicle, to which the opposing glare had an effect and the relative constancy of the effect over a long range of these distances. Just how far a substantial glare effect extends is not known, but the results of Study 2 indicate that visibility will be affected to a considerable extent by opposing headlights at distances in excess of 3,000 feet.

That human beings differ in visual ability and sensitivity to glare was known, but the effect these differences might have on the relation between target detection distances and lateral separation were highlighted by the studies. These differences are shown (fig. 4) by comparison of the curves for the three subjects (N) and the two subjects (L). It was surmised that the reason for this difference in the relative effect of glare is that the two groups of subjects (differing in initial visual ability) were at different distances from the glare car when the target was at threshold for them individually and they were, therefore, subjected to different levels of glare when detection was made. If targets of different, initial (no-glare) difficulty had been used, the result would have been further differences in the relation between detection distance and lateral separation.

Analysis of Methods

Assuming that target detection distance is the criterion chosen for evaluating the effects of glare, recommendations can be made as to the study method on the basis of the first two studies in which target detection distance was the parameter. Figure 10 illustrates the several theoretical relationships between target detection distance and distance of the glare car for three different methods of study. The broken-line curves represent hypothetical, typically shaped curves of target detection distance by glare car distance for constant lateral separations. The two curves in each graph represent data for different lateral separations for different targets or for test subjects having different sensitivities to glare. The light, solid lines are intended to illustrate the manner in which the data would be derived for the particular study method under consideration.

The top part of figure 10 illustrates the relation of target detection distance to distance of glare car and the manner in which the data were derived for Study 1, in which the opposing glare car and the target were stationary and the subject drove toward the target. The result of this fixed relation between glare car and target was a fixed relation between distance of the glare car and target detection distance. As shown in figure 5, the data for the different lateral separations all fell on a diagonal line of unit slope that intersected the ordinate axis at a distance equal to the longitudinal separation between the glare car and the target. This demonstrated that, for any

[5] The British studies utilized theoretical expansions of field data. In part, they showed, for British high and low beams and for appropriately aimed American low beams, curves for seeing distance—for straight-ahead and curb-side test objects—that fell gradually with decreasing distance of the glare car and then rose slightly at small distances. It is significant that the curve shapes and the minimum seeing distances varied with different positions of the object across the roadway and with different headlight beam configurations (high beam facing high beam or low beam facing low beam).

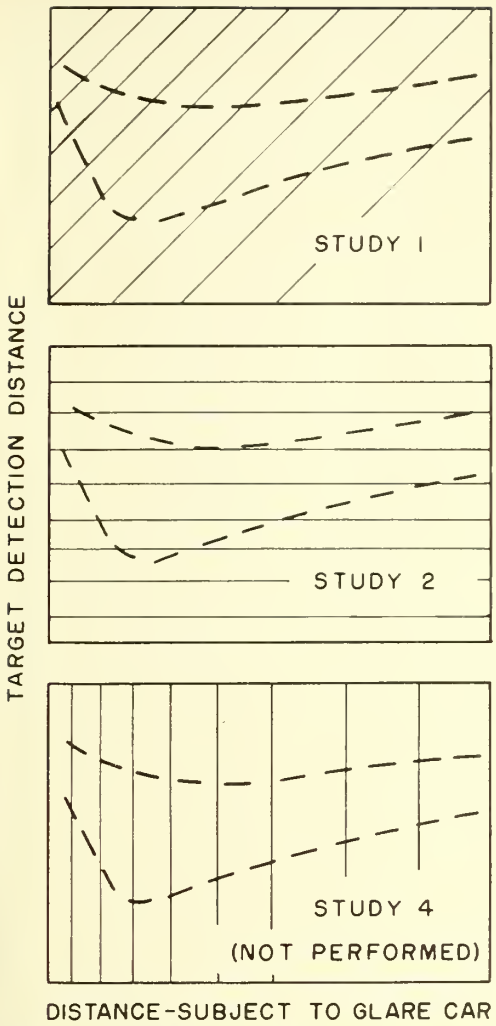

Figure 10.—Theoretical relations between target detection distance and glare car distance for different study methods. Curves are hypothetical and light solid lines show generation of data.

one setup, a direct relation existed between the distance of the glare car and the distance at which the target was detected. Therefore, the general value of Study 1 was severely limited because all data fell along one of the diagonal lines in the top of figure 10. However, this limitation could be overcome by collecting data for several setups, each of which would involve a different distance between glare car and target. The data for each setup then would fall on one of the diagonal lines in the top graph of figure 10 and a curve could be drawn through the appropriate points for each lateral separation.

In Study 2, subject and target were both stationary, and the glare car moved. With this method, distances of the glare car were determined for constant target detection distances. The data, therefore, fell along lines of constant target detection distance, as shown by the horizontal lines in the middle graph of figure 10. The threshold points that were obtained from the intersection of the subject's line of travel with the threshold curves (fig. 8) were used to generate the curves of target detection distances by glare car distance (fig. 9). Because the threshold curves extend for such long distances and are nearly parallel to the line of travel, the locations of these points of intersection are subject to large errors. The fact that these curves are nearly parallel to the line of travel indicates that the glare effect was apparently fairly constant for most of the longitudinal distance between subject and glare cars. Use of the Study 2 method would be suitable where target detection distance changes rapidly with glare car distance, illustrated by those parts of the target detection distance curves that have considerable slope. However, reference to figure 9 shows that this condition of rapid change would apply to only a small part of the curves.

A third variation in methodology, which was not performed, would be to have the test subject and the glare car stationary while the target is moved toward the subject. This method is shown as Study 4 in figure 10. With the glare car and subject being stationary, the glare condition would be constant and the distance at which the target could be detected at different lateral separations could be determined for constant glare. Several different distances between the glare car and the subject would be used. The data would then fall along lines of constant distances between subject and glare car and would be represented by the vertical lines in the bottom part of figure 10. This would probably be the best method for obtaining the target detection distances for the parts of the curves that have a shallow slope. For the steeper slope parts of the curve, the points of intersection would be subject to error. The main objections to this study method are the difficulty of moving the target toward the subject and the consequent changes in the environment of the target.

RECOMMENDATIONS

Method

Although the greatest initial objections were made to the method of Study 1, the use of several relative distances between target car and glare car disposes of these very objections. This method, with the following-described modifications, is therefore recommended for visibility tests based on target detection distance.

In addition to using several relative distances between glare car and target, many test subjects should be used and the choice of subjects preferably should represent a cross section of ages and different degrees of visual ability. There is evidence that older drivers have poorer night vision (*6*) and are affected by glare to a greater degree than those having normal vision (*7, 8*). Note: Vision scores under daylight levels of illumination are not an assurance of the degree of a person's night visual ability (*9*).

The use of one particular target for glare tests would give relations between lateral and longitudinal separations and detection distance for that target only. These relations might be different for different targets, depending on the magnitudes of the distances at which the targets were detectable and at which the different subjects would be able to detect them. Therefore, it is recommended that several critical targets of interest and of varied difficulty be used.

Changing the position of the target transversely across the driver's line of travel is not recommended because this will change the illumination on the target from the driver's own headlights and will change his line of sight with respect to the opposing headlights. It will be shown in the discussion of Study 3 that the background against which the target is viewed has a critical bearing on the detectability of the target. A possible effect caused by light from the opposing headlights reflected from the pavement in the vicinity of the target is also to be discussed. Light from the opposing headlights may be reflected from the target itself. The results of a study by Schwab (*11*) show that, aside from the glare effect, light from opposing headlights has other effects on the visibility of targets. Changes of target position, therefore, may be equivalent to using different targets and, if done, a complete set of data should be taken for each position and analyzed separately. It may be that minor changes in target position across the driver's lane may not be critical when the target can be detected at distances of 500 feet or more. Still, any change in target position will tend to increase the variability of the data.

Because threshold is related to the probability of detection, target detection will be affected by the degree of attention devoted to the task and the degree of expectation associated with the target's presence and location. According to Roper (*10*), objects were detectable at twice the distance when drivers were actively looking for them as opposed to unexpected targets. It is therefore important that all subjects receive the same instructions and use a constant criterion such as "I definitely see it," or "I think I see it," for all tests so that the results can be compared. To reduce guessing and anticipation during the experiment, it is suggested that the target not be present at all times and that the targets be changed at random between runs.

Because it is difficult to determine beforehand where the critical points in the curves exist, such as breaks in the curves and minimum detection distances for each lateral separation, particularly when several targets differing in difficulty are used, preliminary estimates of the relative distance between glare car and target and the lateral separations to be studied should be based on pilot studies in which the same targets are used and the same subjects participate.

Greater efficiency may be obtained in executing the field studies if several test subjects are employed simultaneously. Allowance should be made for any differences in their lateral positions. One possibility would be to have them alternate positions. They could alternate driving and riding in a carefully balanced and random pattern. No

subject should be in the back seat of the test car because sometimes the opposing headlights will be blocked by those in the front or by parts of the car itself. One possible disadvantage to this procedure would be the differences in the degree of concentration if one participant drove while the others were able to devote their full attention to looking for targets.

To reduce reaction timelags and promote greater accuracy in the distance measurements, some convenient method of instrumenting the distance measurement and of recording responses should be used; for example, a fifth wheel device for distance measurement and a pushbutton that could be used to record responses by actuating a printout or pen recorder. Regardless of the method used, test subjects must be prevented from influencing each other when more than one is present for a run.

The exploratory studies reported here and studies reported elsewhere have been limited to the situation in which a single opposing vehicle is met. Although this is not an unusual situation, it is perhaps more common for a driver to be faced by several opposing vehicles at a time and at frequent intervals along the highway. It is therefore suggested that the more critical situation of a continuous line of opposing vehicles be tested. The spacing of these vehicles and the number of lanes of opposing vehicles could be varied to simulate different volume conditions and lane configurations. Then, the variable—the distance of the opposing glare car—would be replaced by the level of glare in terms of density of opposing vehicles or veiling brightness. (Veiling brightness is discussed in connection with Study 3.) Inasmuch as the first two exploratory studies have shown that glare effects extend for long distances from the opposing glare car, the line of opposing vehicles would have to extend for considerable distances. Should this not be feasible, shorter lines of opposing vehicles could be utilized, but another variable would be present; that is, distance to the first or last opposing vehicle. It is possible that the difference in glare effect caused by a continuous line of opposing vehicles and that caused by some critical segment of the line may be negligible. To determine the critical segment, if there is one, might require an extensive study.

Realistic Conditions

One further suggestion is made. Because studies of headlight glare made when the cars have clean windshields do not achieve typical realism, perhaps the most critical test should include a car having a dusty windshield. However, problems can be foreseen in trying to maintain a constant condition of dirtiness of the windshield; but perhaps, some method might be developed to simulate the light-scattering properties of a dusty windshield.

Detection Distance as a Parameter

The measurements of target detection distance given here are the maximum possible; they represent measurements at the lowest possible limit of performance—bare detection under conditions in which the test subject is concentrating on detecting something, usually knows what he is looking for, and knows approximately when and where to look for it. These are not realistic conditions for the actual driving situation and compared to the test situation, the subject will be closer to the target when he detects it. At detection, the angular dimensions of the target, visual angles subtended by the visible dimensions, will be greater and the illumination on the target will be greater by the square of the ratio of the respective distances (inverse square law of illumination). For example, if the subject needs to be half as close to detect the target in an actual driving situation as in the test situation, the angular dimensions of the target would have to be doubled and at the same time the illumination on the target would have to be increased four times. By the same token, relative detection distances do not represent the same relative degrees of visibility. It cannot be said that for one set of conditions *visibility* is double that for another set, but only that the *detection distance* for a *particular target* is twice as good. However, in a test situation, the degree of attention and expectation existing in the actual driving situation cannot be simulated, and if usable responses are to be obtained some concessions must be made to practicality.

The detection distance obtained by taking the average of a number of detection distance observations represents the distance at which the target was barely detected 50 percent of the time when maximum concentration existed. Because relative results are useful, this would be acceptable, if actual driving detection distances were directly proportional to those obtained in the test situation. This appears doubtful, however, because of all the other changes that take place as distance to the target changes. To partially adjust for realism, it is suggested that some higher probability level of detection be shown, such as those distances at which the target is detectable 95 percent of the time. Practical limitations, in terms of the amount of data that would have to be collected, would prevent the achievement of probabilities of detection of more than 95 percent. When it is possible to obtain approximately 100 replications for each condition (one lateral separation, one target, and one setup of glare car and target) the detection distance that is exceeded 95 percent of the time may be taken from a cumulative frequency plot. This procedure is unreliable when the sample size is much smaller than 100 replications. For example, when a sample size is 20 observations, the lowest fifth percentile would have to be based on only one observation. When the sample is smaller than 100, a normal distribution may be assumed or tested for, the mean and standard deviation calculated, and the fifth percentile computed. The larger the variability in the data, the larger the sample size will have to be for reliability.

The problems of relating the results of visibility tests to the real driving situation arise from the use of detection distance as the parameter for ascertaining the effect of opposing glare on visual performance, because the only data that can be obtained are at locations where the target is at threshold. The level of visual performance at other locations cannot be measured. That is, if, under a given set of conditions, a target of interest can be seen—it is above threshold—there is no way of knowing how *well* it can be seen.

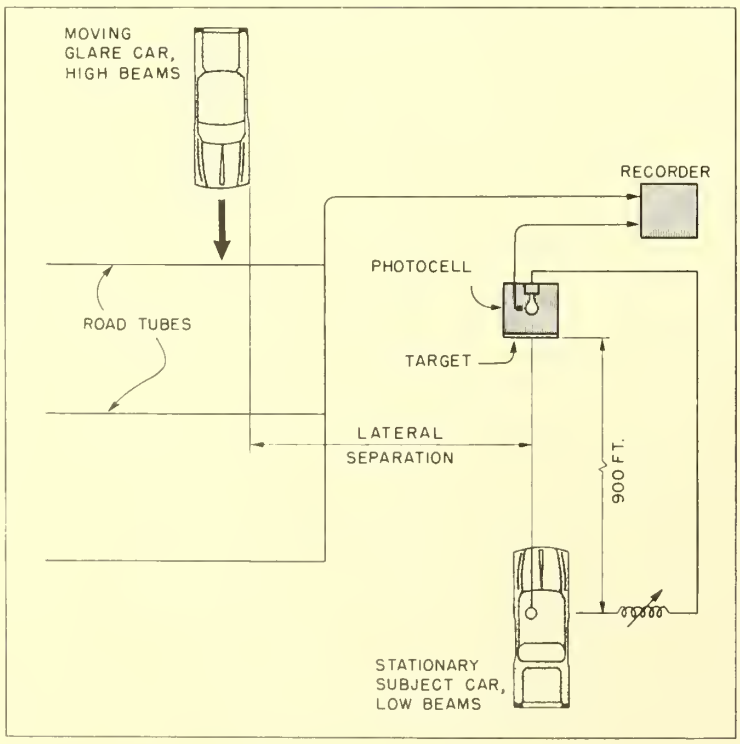

Figure 11.—Field layout for Study 3.

The recommendations for this type of study hold only for the assumption that these effects on target detection distance are the criteria for evaluating the effects of headlight glare. This is another way of saying that studies of target detection distance should be performed only if effects on target detection distance are meaningful, which is questionable. Discussion of some of the other controls and factors of importance in visibility tests, such as measures of glare levels and the effects of the areas against which the target is viewed, must be postponed until after the discussion of the third study.

STUDY 3

The analyses and interpretation of the results of Studies 1 and 2 were plagued by the problems of the dependency of the results on a particular practical target, of possible variations caused by different headlight aims and different positions of the target, and by the simultaneous changes in both the illumination on and the angular size of the target as the driver approached it. In order to obtain a general picture of the extent of the glare effect, a different approach from the previous methods was used in Study 3.

The aim was to have a fixed target of constant size, and to change its visibility by varying its brightness. It was thought that the relative brightnesses necessary to maintain a constant level of visibility of the target would give indications of the relative effects of different lateral separations and of longitudinal distance of the opposing vehicle. The only measure of the level of constant visibility that could be obtained was the threshold level.

The field layout for Study 3 is shown in figure 11. The target was a 21- by 26-inch white translucent plexiglass screen illuminated from behind by an incandescent lamp. A variable transformer was used to vary the voltage across the lamp and thereby vary the brightness of the target. The voltage induced in a photocell mounted next to the lamp was recorded. A range of brightness readings on the target was obtained by using a Spectra brightness spot meter (*12*) reading directly in footlamberts (lumens/sq. ft.); this calibration was used to transform the recorded photocell output into terms of brightness.

The subject was seated in a stationary car and viewed the self-illuminated target at a distance of 900 feet; the target was situated directly ahead and its center was approximately 3 feet above the pavement. The subject's eye height was 4 feet: the line of sight was, therefore, essentially parallel to both the path of the glare car and to the pavement. The 21- by 26-inch rectangular target, at 900 feet, subtended visual angles of approximately 7 by 8 minutes of arc. The subject's car headlights were on low beam to establish a constant brightness on the pavement and adjacent area that is typical in the normal driving situation.

As the glare car, with high beams on, approached at a speed of 10 miles per hour, the subject used the variable transformer to adjust the brightness of the target so that it remained approximately at the threshold of visibility. He did this by increasing the target brightness until he could just detect the target, then decreasing the brightness until the target disappeared, etc. After some practice he could do this rapidly and with very little hand movement. The resultant oscillations on the recorder tape were at a frequency of approximately 1 cycle per second, and the oscillations were of low amplitude. The average of the peaks and troughs of the oscillations was taken as threshold.

After the glare car had passed him, the subject continued to keep the target at an approximation of threshold while his eyes readapted to the no-glare condition. This was assumed to have occurred when the record of the photocell output reached a constant level. The average of these outputs for all runs for each individual was taken as his no-glare threshold. The longitudinal distance between the subject and the glare car was obtained by means of road tube actuations that were recorded simultaneously with the photocell output.

Runs were made in random order at lateral separations of 7, 20, 32, 57, and 107 feet. The first road tube was located 3,000 feet from the subject, but because it apparently took some time for him to find his threshold level, the data collected for distances of more than 1,800 feet are not considered entirely reliable.

Results

It had been expected that the results of Study 3 would be more clearly interpretable than those of Studies 1 and 2; it was believed that the variables were more exactly controlled and could be measured more exactly. This was partially true, at least in a qualitative sense, and is illustrated by data shown in figure 12. A set of curves is shown for each of two subjects, A and B. The part of the abscissa to the right of the zero distance

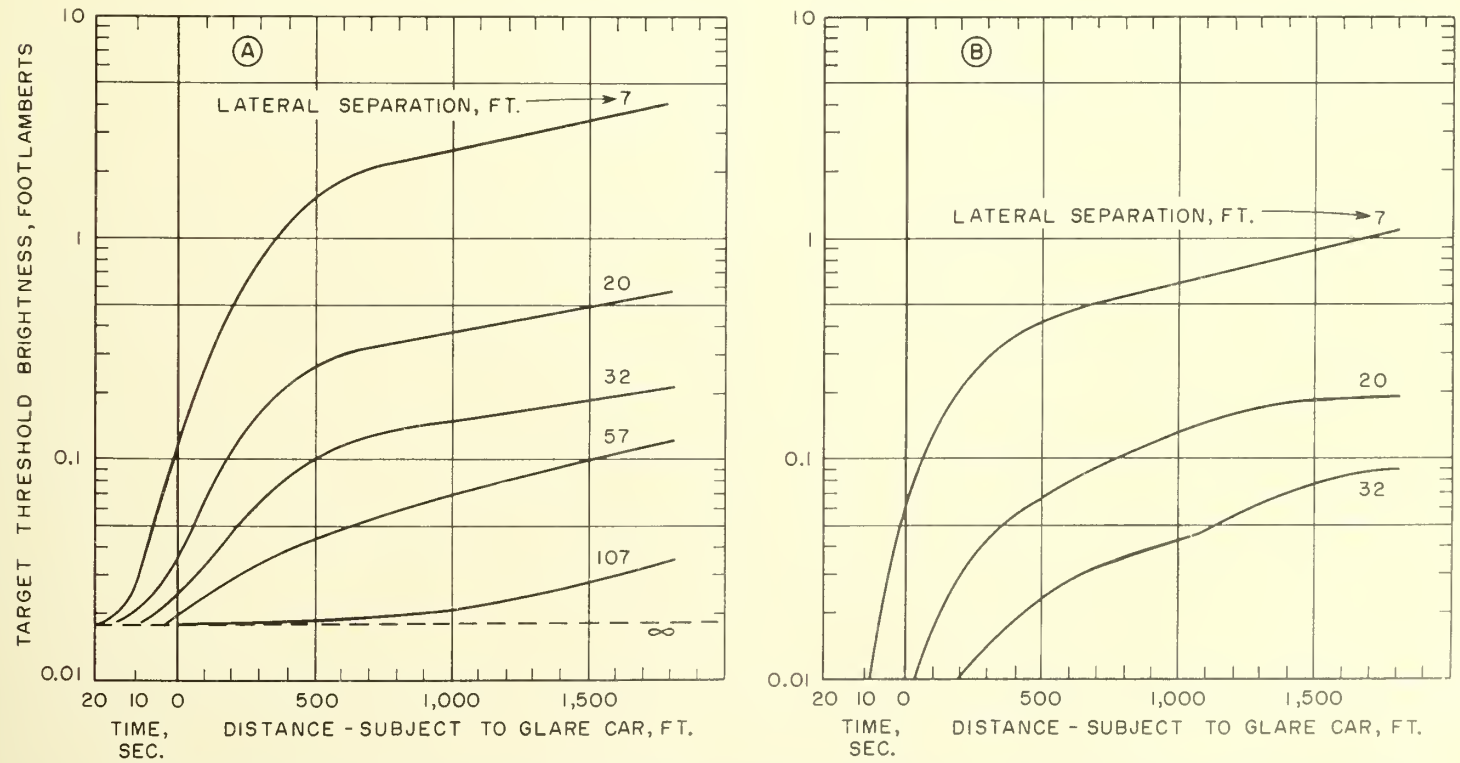

Figure 12.—Threshold brightness of 8-minute target; variation by distance of glare car for different lateral separations for two subjects, A and B (Study 3). Time scale equivalent to distance scale for 10-m.p.h. running speed of glare car.

point shows the longitudinal distance between the subject and the glare vehicle; the part to the left shows the time after the glare car had passed. (The time scale shown is equivalent to the distance scale for the 10-m.p.h. running speed of the glare car; that is, 10 m.p.h. is 15 feet per second, and the time and distance scales have been drawn so that a 1-second interval on the former is the same length as a 15-foot interval on the latter.) The ordinate shows the target brightness in footlamberts when the target was at the threshold of visibility for each of the two participants. Each curve is for a different lateral separation and represents the smoothed average of at least three and sometimes four replications.

The interpretation of figure 12 can be visualized, as follows: consider the subject to have been situated at distance zero, facing the glare vehicle as it approached from the right of the figure. As the distance from glare source decreased, the decline in the curves indicates that the brightness necessary to maintain threshold visibility also decreased; that is, the disabling effect of the glare source was apparently decreasing. This was contrary to what had been expected.

For example, in figure 12, part A, for a lateral separation of 7 feet, when the glare vehicle was 1,800 feet away, a target brightness of 4 footlamberts was required for threshold visibility; when the glare vehicle was 600 feet away, only 2 footlamberts were required; and when the glare vehicle was not present, only 0.018 footlambert was required. The fact that all the curves do not approach the no-glare level at zero distance doubtless was the result of the subject's eyes not having readapted to the no-glare condition. The time required for this readaptation is shown to the left of the zero distance point.

These readaptation times are possibly longer than those that occur in the normal driving situation because of the length of exposure. But, the readaptation times obtained also might have been shorter than those in the normal driving situation because of the slowness of the simulated meeting. The driver obviously must be readapting during the time the glare level is dropping as the opposing vehicle is about to pass him. This time period between exposure to the higher glare levels and disappearance of the glare source would be much shorter at normal speeds; readaptation in the normal driving situation would start later and might, therefore, continue for a longer period of time after the glare car had passed the driver. The net effect on the observed adaptation times of these two compensating operations cannot be determined for the test situation of Study 3.

Because the curves represent the smoothed averages of a few replications, there might have been significant changes in slope that could not have been determined because of the limited sample size. As an indication of the variance of threshold brightness among runs—or of the range of brightnesses that approximated threshold—participant A's no-glare thresholds ranged from 0.013 to 0.026 footlambert, or one-third of a log unit. This is comparable to the range in threshold measurements normally obtained in this type of research. Therefore, the no-glare threshold measurements had the least variance among runs of any of the other data collected.

Both subjects of Study 3 had taken part in Study 1: A was one of those who had shown relatively poor nighttime visual performance (*L*) and B was one of those who had been classed as having normal vision (*N*). As shown in figure 12, A required more than half a log unit more brightness to detect the target than B. In order to bring the target down to threshold when the glare car was at the wider lateral separations, participant B required lower brightnesses than could be obtained with the apparatus available. Furthermore, for the no-glare condition he was able to detect the target, without its being internally illuminated, by the light from his own headlights.

Psychophysiology of Vision

An explanation of the results obtained in Study 3 requires examination of the phenomena that are operating, based on a background of the psychophysiology of vision (*13, 14*).

When one looks directly at an object, light from the object forms an image at the central part of the retina, referred to as the fovea, which is the rod-free area of most distinct vision. Light entering the eye from a bright source away from the line of sight should form an image on the retina away from the fovea. However, because the media of the eye (cornea, lens, etc.) are not perfectly transparent, this light is partly scattered within the eyeball and some of it falls on the fovea, thus raising the adaptation level. The effect is similar to the interposition of a veil of light between the object and the eye and has, therefore, been termed veiling brightness. An equation for the veiling brightness (B_v) produced by a point source such as a distant headlight is given by Fry [6] (*15*):

$$B_v = \frac{kE\cos\theta}{\theta(\theta+1.5)} \qquad (3)$$

$$E = \frac{I}{d^2} \qquad (4)$$

Where,

B_v = veiling brightness, footlamberts;
E = illumination at the eye, footcandles;
I = intensity of the source directed at the eye, candelas;
d = distance of the source from the eye, feet;
θ = angle between the source and the line of sight, degrees;
k = proportionality factor, 28.9 when the preceding units are used.

[6] This equation was derived from experiments in which male college students participated. Older drivers might be expected to experience more scattering because of the increased opacity of the media of their eyes (*8, 16*).

An object is detectable because of the contrast of its brightness (B_o) to the brightness of the background (B_b) against which it is viewed (more correctly, to the level of brightness to which the area of the retina adjacent to the image is adapted). Contrast is expressed by Blackwell (*17*) as:

$$C = \frac{B_o - B_b}{B_b} \qquad (5)$$

Because the veiling brightness exists in the eye, its effect is to appear to add to both the brightness of the target and of the background. However, it decreases the effective contrast (C_e) because the brightness difference between the object and its background remains unchanged, but the adaptation level (the original B_b plus the superimposed B_v) is raised.

$$C_e = \frac{(B_o + B_v) - (B_b + B_v)}{B_b + B_v}$$

$$= \frac{B_o - B_b}{B_b + B_v} \qquad (6)$$

Therefore, the extent to which a given magnitude of veiling brightness will reduce visual ability will depend on the magnitudes of the existing brightnesses. To further complicate matters, lest it be thought that the contrast required to detect a given size target is constant, threshold contrast varies with adaptation brightness.

Target Detection and Veiling Brightness

From equations (3) and (4) and a candlepower diagram for the configuration of headlamps (system of four lamps set on high beams) used in Study 3, values of veiling brightness [7] caused by the opposing headlights were calculated for each of the lateral separations for the geometric conditions of the experiment. The changes in veiling brightness in relation to the longitudinal distance of the glare car is shown in figure 13. The calculations are for a horizontal line of sight and were based on the assumption that the glare car was traveling in a path parallel to the subject's line of sight on a horizontal plane. The magnitude of the veiling brightness and the shapes of the curves will be different if any of the following given conditions are changed: orientation of the line of sight, highway geometry (affecting the orientation of the opposing headlamps), number of glare sources, or headlight beam (high to low). In addition, the degree of opacity of the media of the eye will differ between individuals. The peculiar shapes of the curves in figure 13 are the result of the way in which the factors affecting veiling brightness changed in relation to lateral and longitudinal separation.

[7] However, veiling brightness can be measured directly by means of the Fry-Pritchard glare lens used in conjunction with a Pritchard telephotometer. This lens, attached to the photometer, measures the veiling brightness from all light sources in the field of view by integrating them according to equation (3). Both this lens and the Pritchard photometer are briefly described in reference *12*. Reference *18* contains a detailed description of the lens.

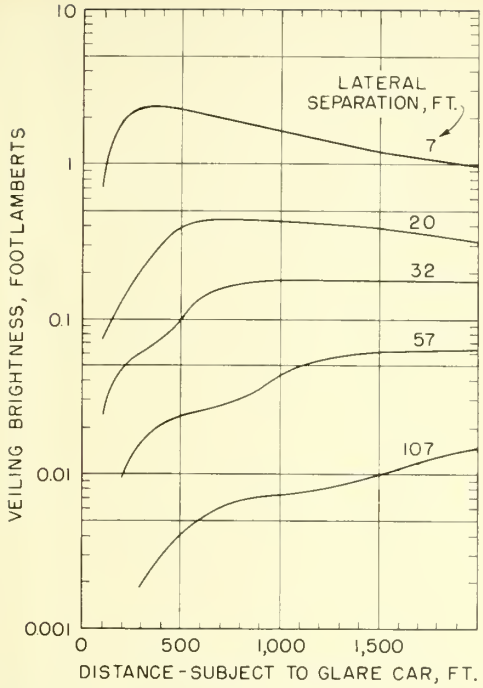

Figure 13.—Veiling brightness calculated from equation 1 for opposing high beams for conditions of Study 3 (line of sight parallel to pavement and to path of glare car). Each curve is for a different lateral separation.

The curves in figure 13 were drawn in the same format as those of figure 12; that is, the subject is assumed to be situated at distance zero facing the glare car as it approaches from the right of the illustration. Veiling brightness at the 7-foot lateral separation rose gradually to a peak, as the glare car approached from 2,000 feet to within 400 to 300 feet, and then dropped sharply. Veiling brightnesses at the 20- and 32-foot lateral separations were fairly constant for long distances and began to drop off at about 600 and 700 feet, respectively. Veiling brightnesses for the 57- and 107-foot lateral separations were already decreasing as the glare car approached to within 2,000 feet of the subject.

Candlepower (I) used to compute the illumination (E) which is then used in the veiling brightness equation (equation 3) are not shown for large angles on the available candlepower diagrams. At short distances, candlepower is very low and the glare angle (θ) is getting so large that veiling brightness becomes very small. Veiling brightness must obviously be zero at zero longitudinal distance of headlights from the subject; therefore, the curves must approach the zero distance line asymptotically on the log scale. Target threshold brightness is related to veiling brightness through the brightness-contrast function, equation (6). Unfortunately, the equipment available at the time this experiment was performed was inadequate to measure the low levels of background brightness that were present at the site, so target contrast could not be calculated. However, *all other things being constant,* target threshold brightness should vary directly with veiling brightness; for example, where veiling brightness is high, target brightness should be correspondingly high.

Inconsistencies Noted

Comparison of the threshold brightness curves of figure 12 with the veiling brightness curves of figure 13 shows that generally the brightness required to keep the target at threshold changed in much the same manner as the veiling brightness. However, there was some inconsistency: for the 7-foot lateral separation, between 1,800 and 300 feet, target brightness was falling as veiling brightness was rising. At lateral separations of 20 and 32 feet, target brightness was falling even when veiling brightness remained fairly constant. Only for the 57- and 107-foot separations did target brightness and veiling brightness decrease concurrently.

A comparison of the Study 2 target detection distance curves (fig. 9) with the veiling brightness curves from Study 3 (fig. 13) shows them to be generally consistent (to visualize this, imagine that figure 9 is turned upside down). For the narrow separations, at glare car distances where veiling brightness was increasing, target detection distance was decreasing, and vice versa. Therefore, the Study 2 target detection distance curves seem to show a different relationship between the effects of headlight glare and position of the glare car than do the target threshold brightness curves of Study 3. Based on the target detection distance, the disabling effect of glare was shown (Study 2) to increase as the glare car approached from far distances, reach a maximum, and then decline as the glare car came abreast of the subject. But, the Study 3 data on target threshold brightness show the disabling effect of glare to be worst at far distances of the glare car (up to 1,800 feet) and to decline as the glare car approached. Further analysis was made of data from Study 3 to investigate whether the differences in the results were the result of differences in the targets and study methods.

It was postulated that if all other things remained constant for different locations of the glare car, then changes in veiling brightness alone should determine changes in the contrast conditions and, consequently, in target threshold brightness. It should follow, therefore, that, when the same veiling brightness occurs for different locations of the glare car, the corresponding target threshold brightnesses should also be equal.

To check the validity of the postulation, the data of figures 12 and 13 have been combined in figure 14 to show target threshold brightness for the associated veiling brightness for each driver, lateral separation, and distance of the glare car. Two sets of curves are shown, one for each driver. The small

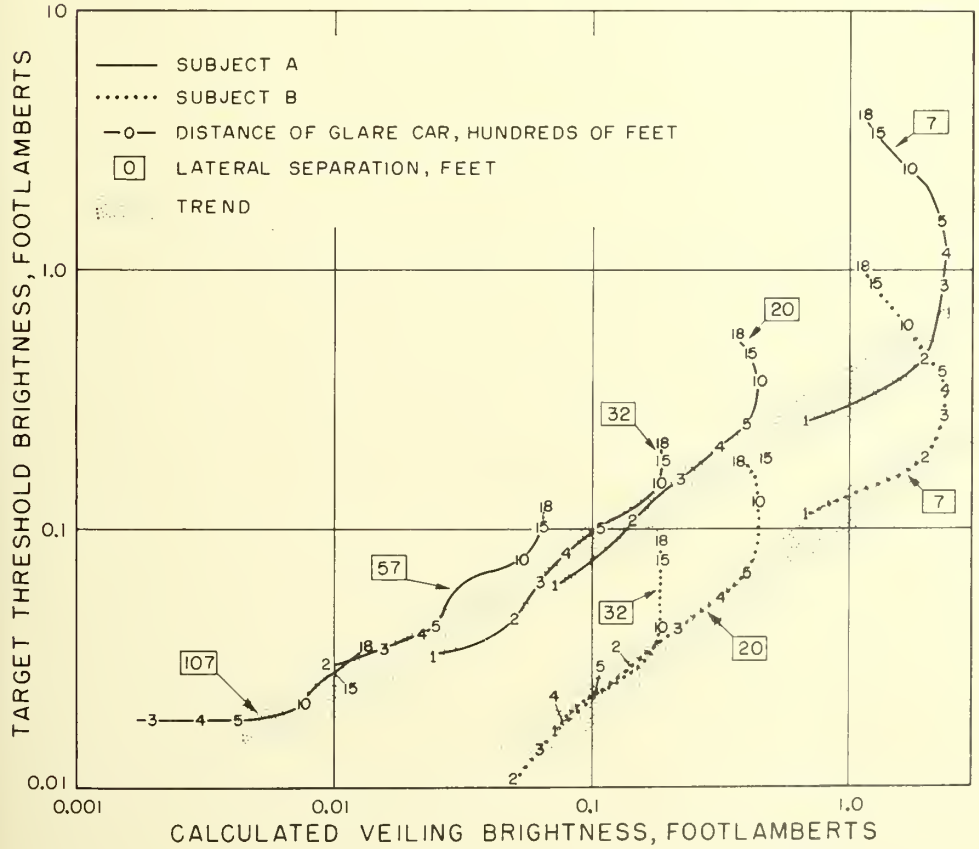

Figure 14.—Relation of target threshold brightness to calculated veiling brightness for two subjects.

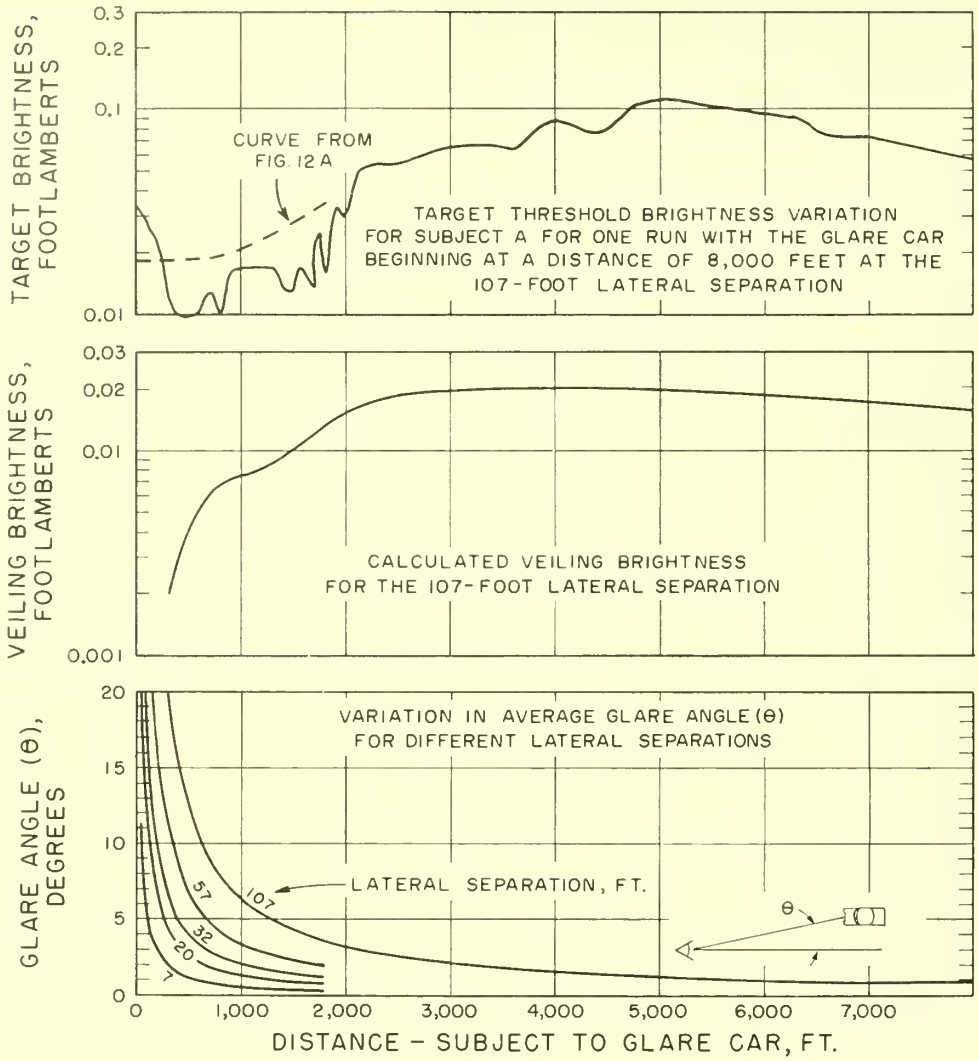

Figure 15.—Variation of target threshold brightness, calculated veiling brightness, and average glare angle (θ) by glare car distance.

numerals on the curves are the distances, in hundreds of feet, of the glare car at separations for each curve. If the target threshold brightness is dependent only on the level of veiling brightness and if the calculated veiling brightnesses are correct, all the curves for each subject should overlap. Considering the limitations of the target threshold determinations and the fact that veiling brightness has been calculated rather than measured, the coincidence of the curves for the different lateral separations for each subject is very good. An arbitrary trend line has been drawn for each driver. The curves for each lateral separation have been drawn as far as the available values of veiling brightness would permit. Had very low values of veiling brightness been available for the short distances, all the curves theoretically would follow the trend lines to the no-glare level were it not for the lag in adaptation. This lag may account for the decreasing slope of most of the curves at the short distances.

Possible sources of inconsistencies

Although most parts of the curves in figure 14 show a high degree of coincidence, some discrepancies occur: some parts of the curves do not follow the trend lines or the trend slopes. The inconsistencies noted when the target brightness curves of figure 12 and the veiling brightness curves of figure 13 were compared stand out clearly. These departures from the trend lines occur most pronouncedly at the narrow separations and at the long distances from the glare car.

Four conditions that may have been the source of the inconsistencies in the data collected are discussed in the following paragraphs. (As an aid, the reader should attempt to visualize the target and its background, as if he were the test subject.)

• Part of the target, being somewhat closer to the pavement than the subject's eyes, was viewed against a background of pavement. The opposing headlights not only introduced veiling brightness into the eye but also lighted up the pavement near the glare car. In addition, at long distances there was some specular reflection from the pavement between the glare car and the subject. Furthermore, for long distances and, particularly, at narrow lateral separations, the opposing vehicle was very close in angular distance to the target. Therefore, for these long distances and narrow separations, a lighted area of pavement existed close to the target. This would have decreased the contrast for the target, and a higher target brightness would have been required to bring it up to threshold.

• The veiling brightness calculations have been based on the assumption of a fixed, unwavering line of sight. Presumably the target was approximately at threshold; therefore, it should not have been visible for approximately half the time and so could not be fixated upon constantly. (Even if the threshold brightnesses derived are not for 50-percent detection, the oscillations in target brightness caused by the driver required that the target be undetectable approximately half the time.) Small eye movements may have intermittently brought the line of sight closer to the opposing headlights, thereby increasing the veiling brightness to more than had been calculated. It is possible that the test subject may have occasionally glanced directly at the headlights.

• According to Schwab (*11*), forward scattering in the atmosphere of the light from the opposing headlights is appreciable at small angles. This scattered light would be similar in nature and effect to the scattered light in the eye that produces veiling brightness and would reduce the effective contrast of the target, and thus a higher brightness of target would be required for threshold.

• It is also possible that the extremely bright headlights, being the only prominent objects in the field of view, tended to partially distract the subjects' attention and disturb their concentration on the target; as a result the target had to be brighter. The effects of all the conditions described would be present to some extent in the actual highway driving situation. The effect of the spillover light from the opposing headlights onto the pavement of the driver's own roadway would be mediated by the characteristics of the median and the geometries of the two roadways.

The questions raised previously—as to why the target detection distance curves from Study 2 data showed a somewhat different relation between the effects of headlight glare and location of the glare car than do the target threshold brightness curves developed from Study 3 data—therefore cannot be answered exactly. It is possible that the answers may lie in some of the differences between the studies. For Study 3, the differences between the shapes of the curves for calculated veiling brightness and target threshold brightness were attributed to several possible factors. Among these were the reduction in target contrast caused by the area of pavement lighted by the opposing vehicle and the proximity of the opposing headlights to the target. However, in Study 2, part of the

background against which the reflectors (the target) were viewed was the dark car body and not the pavement. In addition, the target car was offset laterally from straight ahead (the angular distance of the reflectors on the right side of the car averaged about one-half degree away from the straight-ahead position).

8,000-Foot Run

The target threshold brightness measurements discussed so far have been for glare car distances of 1,800 feet or less. The top part of figure 15 is a plot of the target threshold brightness for subject A for one run—the lateral separation was 107 feet and the glare car started at a longitudinal distance of 8,000 feet. The glare car moved at 60 m.p.h. to 3,000 feet, then was rapidly decelerated to reach a speed of 10 m.p.h. at 2,400 feet. A sag grade change in the runway of 0.3 percent existed at a distance of 2,650 feet from the subject. The fact that the curve for the small longitudinal distances fluctuates, and also falls below the average no-glare threshold target brightness of the other runs for the 107-foot lateral separation, should not cause too much concern. Target threshold brightness fluctuated during the no-glare condition and, for this particular run, happened to be low. To illustrate some of the variance in the target threshold brightness, the curve has not been smoothed very much; only the minor oscillations around threshold have been omitted. The fluctuations shown in the top part of figure 15 are probably random and do not represent any significant changes in threshold.

As illustrated, the target had to be maintained at a substantial brightness while the glare car was between 8,000 and 2,600 feet from the subject. In other words, substantial glare existed even when the glare car was 1½ miles away. The curve of target threshold brightness from figure 12, part A, for the same subject and lateral separation, is shown for comparison. Were this curve to be extended, it appears that it would meet the 8,000-foot curve at about 2,100 feet.

For much of the 8,000-foot distance the target brightnesses recorded for subject B for this same type of run were too low to be measured. However, from 8,000 to 3,000 feet the measured target brightness was about 0.01 footlambert; a slight rise above 0.01 footlambert was measured between 6,000 and 3,000 feet; this seems to indicate a rise and decline in threshold similar to that for subject A at these distances.

For the 107-foot lateral separation, the middle part of figure 15 shows the variation with distance from the glare car headlights in calculated veiling brightnesses. Although the shapes of the veiling brightness and target brightness curves are similar for both subjects, the target brightnesses for the far distances are much higher than they should be for the corresponding calculations of veiling brightness at these distances. Therefore, it is concluded that the high target brightness thresholds at the far distances were not produced by glare from the headlights alone, but that the effects of reflected light from the pavement, fluctuations in fixation, or forward scattering in the atmosphere also contributed to the results. The lower part of figure 15 shows the variation in average glare angle θ (measured to center of glare car rather than to the individual headlights) in relation to distance for the different lateral separations.

SUMMARY DISCUSSION

The studies reported herein were undertaken with the intention of exploring alternate methods of studying the relationship between headlight glare and median width. Inasmuch as insight into the magnitude and extent of the glare effect and the factors involved was desired, the amounts of data collected were small; therefore, little reliance should be placed on the absolute quantitative results. Furthermore, the quantitative results hold only for the particular conditions studied, such as the geometry of the test situation, target, drivers, and surroundings. The qualitative relations between target detection distance and position of the glare car also were affected by these variables. The tests simulated only one car meeting a single opposing vehicle that had high beams on.

Discomfort

The results of the discomfort test made in Study 1—the only study in which both discomfort and visibility were studied—showed that the data for discomfort caused by glare were too variable, and that discomfort was too difficult to define for a limited study to produce useful results. No relation was apparent between visual and discomfort sensitivities to glare.

Target Visibility

The rear of a black, unlighted car was the target for Studies 1 and 2. The rear, red reflectors were always detected first. A self-luminous target, for which the brightness could be varied, was used in Study 3. The results of all three studies showed that the effects of glare on visibility (target detection) decreased as lateral separation was increased and the effects of the glare were substantial even when the opposing glare car was at a longitudinal distance of several thousand feet. Very limited data from Study 3 indicate that effects of the opposing headlights on visibility may be present even when they are at distances of as much as 8,000 feet from the observer. For any individual lateral separation, the rate of change of visibility related to distance of the glare car was small. Large differences in night visual ability and glare sensitivity may exist between drivers; these differences have been correlated with age by other researchers.

From an analysis of the results of Studies 1 and 2, it was concluded that different relations may occur between position of the glare car and target detection distance for different targets or different drivers. This difference in relation may be caused by the interaction between distance at which detection occurs and level of glare existing at that distance. This occurs because of differences in orientation with respect to the opposing headlight beams at different distances from the glare car.

Methods

The method, which was the one used for Study 1, where the glare car and target are stationary and the subject moves toward the target, provides incomplete information if a single fixed relation between target and glare car is maintained. The stationary glare car and target impose an arbitrary relation between distance of the glare car and detection distance. For future research this disadvantage can be overcome by the use of several different relative distances between glare car and target, and by placing the target both in advance of and beyond the glare car.

The method, where target and observer are stationary and the glare car moves toward the observer, which was used in Study 2, is subject to large errors because the cigar-shaped actual glare disability contours cut the observer's theoretical line of travel at such small angles; many repetitions would be required for any accuracy. In addition, the threshold probability levels are unknown, so it is impossible to determine where to draw the contours in the vicinity of those lateral separations where, for some runs, the target is always detectable for the entire run.

For future research recommendations are made for the performance of detection distance studies, using the modified method of Study 1, including the use of several different types of targets, random orders of presentation, the use of representative observers, and the use of constant criterion for reporting detection (in terms of the observer's confidence of detection); it is also suggested that one of the most critical, realistic conditions would be one in which a driver would be faced by a large number of opposing vehicles on low beam. For a situation even more critical and realistic, a dusty windshield could be used.

The difficulty of interpreting relative detection distances for the purpose of assessing visibility is discussed. This difficulty arises particularly because of the probable lack of proportionality between detection distances under the conditions of attention and expectation existing in the test situation as compared to the normal driving situation. To more nearly approach detection distances that are representative of detection distances in actual driving, it is recommended that the results shown be the fifth percentile detection distances rather than the average. The use of the fifth percentile rather than the average gives distances at which the target is detectable 95 percent of the time rather than 50 percent.

Other Factors Affecting Visibility

In Study 3, subject and target were stationary and the subject attempted to keep the self-luminous target approximately at the

threshold of visibility, while the glare car moved toward him. As expected, target visibility increased as lateral separation increased. At the narrow separations, readaptation times to the no-glare condition were in the order of 5 to 10 seconds. Surprisingly, target visibility appeared to increase as the glare car approached the observer, at least for the distances analyzed (1,800 to 0 feet). This result was consistent with the manner in which calculated veiling brightness changed as the glare car distance changed for the wide separations, but not for the narrow separations. Comparison of veiling brightness and target brightness showed that the calculated veiling brightnesses either were not the same as those present in the experiment or that other factors were affecting the visibility of the target. It was conjectured that these other factors, which may have affected both the observer's adaptation brightness and the contrast of the target, were caused by fluctuations in the line of sight, by changes in brightness of the pavement against which the target was viewed caused by light from the glare car headlights, or by forward scatter in the atmosphere of light from the glare car headlights. Other possibilities are that the observer occasionally glanced directly at the headlights or that the headlights, being the only prominent objects in the field of view, had an attention-distracting influence.

Therefore, a report of the location of the opposing vehicle alone is not sufficient to define the visibility conditions. The independent variable should be some measure of adaptation, rather than the number or position of opposing vehicles, or horizontal footcandles. In night visibility research, therefore, account should be taken of all factors that might affect the visual adaptation level so that results may be tied to a common denominator.

REMARKS

As a basis for an explanation of at least part of the results of Study 3, some aspects of the physiological basis of vision are repeated here. Visual receptors are sensitive to a wide range of luminances. They adapt to the particular levels available, although there is a timelag, particularly when the luminance is decreasing from higher to lower levels. Objects are discriminated by contrast; this contrast is defined as the difference in brightness between the object and the adjacent background against which it is viewed divided by the background brightness, equation (5).

Because the media of the eye are not perfectly transparent, all light entering the eye is somewhat scattered or diffused. When the field of view is dark, this scattered light from high intensity sources can be of appreciable magnitude in comparison to the brightnesses of objects. This scattered light within the eye has the effect of a veil of light, superimposed over the field of view and it varies in brightness according to angular distance from the source of light. The effect of this scattered light is veiling brightness.

The disability glare effect is primarily the result of veiling brightness, which lowers the effective contrast by raising the adaptation brightness while leaving unchanged the difference in brightness between the object and its background. This is shown by equation (6).

Highway Brightnesses

The brightness of an object, as viewed by the driver, will depend on the light it emits, if any, and the light it reflects from the environment, from the driver's car headlights, from other sources such as other vehicles moving in the same direction as the driver, and from opposing headlights. The brightness of the background against which the object is viewed may also be affected by any or all of these sources. Light on the pavement from the opposing headlights may help delineate the roadway alinement ahead of the driver by increasing the contrast between the pavement and the shoulder. This is especially true when the pavement surface has different specular reflection characteristics from the shoulder; this is usually the situation as roadway shoulders ordinarily are unpaved. In addition, specular reflection from the pavement will enhance the visibility of objects seen in silhouette but will reduce the visibility of objects seen in direct light, such as pavement markings.

Medians

The factors that determine visibility, such as veiling brightness or background brightness, will themselves be affected by median features, such as median width. At any given longitudinal distance separating an opposing vehicle from an observer, a greater horizontal separation will cause the opposing headlights to be at a greater angle from the line of sight; lower intensities of light and, consequently, less illumination will be directed at the eye of the observer. The result of both the increased angle and the lowered intensity will be lower veiling brightness. Background brightness will be reduced because less of the forward scattered component of the light will be present along the line of sight and less light from the opposing headlights will fall on the driver's own roadway.

On divided highways in rolling or hilly terrain, independent roadway design frequently is employed for all or substantial portions of the highways. This type of design is usually combined with curvilinear alinement (continuous flat curves), and the two roadways are designed as separate cross sections having independent grade lines. The width of median between the two roads varies along the highway. In general, this design tends to reduce glare by providing sections of wide medians that frequently are covered by natural growth or dense plantings. Sections of earth or rock may be left in the median area. The independent grades also may reduce the glare from oncoming cars. But sometimes the glare problems may be accentuated where differences in the grades of the adjacent roadways are slight or where their centerlines tend to converge and no barrier to light exists in the median. In good design, such potential problems are located during design and, if possible, avoided; otherwise, screen planting is usually specified.

Screening features

The median features that by interposition ameliorate the effects of opposing headlights can be classed as light obstructors. The opposing headlights can be hidden from view by earth mounds, solid plantings of vegetation, solid fences, or fences that block the view of opposing headlights at almost all angles at which they would be visible—this blocking can be accomplished by a venetian-blind type of slat fence or expanded metal antiglare screen. Partial blockage can be achieved by a chain link fence, which completely blocks the view of opposing headlights at small angles only. Thick and continuous plantings of vegetation in the median can block the view of opposing headlights; less dense plantings may permit intermittent flashes of light from opposing vehicles. Deciduous plants are not as effective in blocking headlight glare when they lose their foliage; this difficulty can be overcome by using evergreens for median plantings. Some limited use has been made of translucent screens that reduce the intensity of the light from opposing headlights by reflecting some, absorbing some, and diffusing the rest.

Other median features may produce undesirable shadows: (1) light from a vehicle may fall on the driver's own roadway; (2) a high curb may put the left edge of the pavement in shadow; (3) a curb may also diminish the contrast between pavement and median; (4) a low solid barrier may accentuate the shadowing effect; and (5) guardrail posts and vegetation in the median may introduce irregular, moving shadow patterns on the roadway ahead of the driver.

Other Solutions

Other solutions or aids to overcome the night visibility problems have been proposed or are in use. Foremost among these has been the increase in visibility obtained by increasing the ambient illumination and roadway brightness by means of highway lighting. But, these luminaires may also be sources of glare.

Additional illumination in the driver's field of view could be provided easily by increasing the intensity and angular spread of the headlights. This would, of course, greatly intensify the glare problem. The headlight glare problem is the result of the need to provide light directed along the roadway while at the same time drivers traveling in opposite directions on the same road face each other's light sources. Polarization of headlights has been proposed as a means of drastically diminishing the apparent brightness of opposing headlights while maintaining or increasing the efficiency of the light output of the driver's own headlights (*19*). This solution has heretofore foundered on the problems associated with the period of transition between partial and complete conversion to polarized

headlights. Changes in conditions since the time this system was last considered may yet enable this solution to be realized (*20*).

A partial remedy for the problem of night visibility on highways has been to increase the reflectivity or luminance of some significant objects of interest in order to make better use of or to overcome the limitations of existing headlight illumination. Examples are: reflectorization of signs and pavement markings, and the use of taillights and rear reflectors on vehicles.

Night Visibility

Opposing headlight glare is a problem because it reduces visibility at night. But visibility at night, even without glare, is not considered good. Therefore, the visibility reducing aspects of headlight glare are a part of and not different from, except in degree of severity, the entire night visibility problem. The night visibility problem can be defined, superficially, as a lack of sufficient light. But what is it that requires sufficient light and what are the results of the lack of light. In other words, what needs to be seen. This need would depend on what visual information is used by the driver in the performance of the driving task. Unfortunately, knowledge as to these information needs is limited, as is an appropriate definition of the major aspects of the driving task itself. As additional information for definition of the driving task is obtained, it is believed that more definite information can be obtained as to the drivers' requirements for light.

In the meantime, speculation may prove fruitful in suggesting additional lines of approach to the problems. Apparently sufficient light presently is being provided for the visual task to enable the night-driving task to be accomplished. This can be stated with some degree of confidence because, for the most part, drivers do succeed in accomplishing the task. However, all this may prove is that human beings are highly adaptable; perhaps the task could be accomplished with more of a margin of safety and comfort. What appears to be lacking is a reliable quantitative measure of the degree of adequacy of visibility for the night driving task; this in turn is the result of a lack of understanding of what constitutes the task.

Detection or visibility of targets at threshold has been emphasized in previous research, perhaps because this is the easiest thing to measure and appears to have the most direct bearing on the visibility problem. The problem of interpreting detection distances has been discussed previously, as has the limited applicability of absolute threshold data to the actual driving situation.

Several instruments have been developed for use in attempts to assess the visibility of objects at suprathreshold levels: two of these instruments are the visual task evaluator (*12, 21, 22*) and the Finch visibility meter (*12, 23*). However, even the visibility data obtained by use of these instruments must be evaluated by arbitrary criteria. In addition to suprathreshold measurements, these instruments permit study of a large variety of types of targets. For example, observations are not limited to targets that are discrete objects. Selected portions of a continuous target such as a pavement edge may be studied. However, the presence in the roadway of objects that may be struck is rare, as differentiated from objects of interest such as pavement lines.

Consider the following hypothetical set of conditions: (1) The driver is confident that no objects except other vehicles are in his path of travel; (2) all vehicles are sufficiently lighted and marked so as to be detectable even under severe conditions of opposing glare. Even with these conditions, a night visibility problem would exist. The driver would still have to be able to see the roadway to obtain information for steering and lateral position control and for judging the location of other vehicles relative to his path of travel.

There are indications that a driver's judgments for steering and position in lane, as well as judgments of his relative speed and position with respect to other vehicles and objects, are determined by his judgments of angular velocities, and that the distance ahead at which these cues are detected is proportional to his speed (*24*). If it is therefore necessary for the driver to detect these cues at some distance ahead, he would be required to reduce his speed under conditions of poor visibility. Should he desire to maintain the same speed as under good visibility conditions, some compensation probably must take place and he may have to devote more concentration and attention to the task of retrieving information. The result may be increased tension and fatigue, but little or no measurable effect on gross driving performance. It is also possible that the driving task might be easier if the driver could determine the course of the roadway farther ahead than is necessary to detect steering cues; thus the driver would be relieved of part of the vigilance task.

On the basis of subjective experience, it seems that, even where the glare effect on visibility is small, such as where a wide median exists, the presence of opposing headlights is annoying. Allied to the problem of the increased vigilance required when visibility is poor is the possibility that the awareness of a deficiency in visibility and the ensuing uncertainty of detecting cues may themselves induce tension. Because man is by nature a daytime animal, the mere presence of darkness may be psychologically depressing.

A driver's attention from the primary visual task of driving may be distracted by opposing headlights or other sources of high intensity illumination such as luminaires, advertising displays, or lights associated with roadside business establishments because they tend to be the brightest and most prominent objects in the field of view. This would be in addition to the decrease in attention stemming from the effect of glare on the contrast of the task. The result of this distraction (noise) could be reduction in visibility caused by the psychological effect in addition to the physiological effects. As another aspect of the problem of the intensified vigilance required, these additional light sources could add to tension and fatigue. In addition to the psychological stress and discomfort engendered, continued exposure to bright headlights may be physically discomforting.

Perhaps the major benefit to be gained by the elimination of high intensity glare sources from the drivers' field of view will be in the drivers' comfort. Therefore, increased emphasis may be warranted on investigations of the relationships between glare and visibility and their relationship in turn to comfort and fatigue.

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New Publications and Films

Introduction

A new publication and two new 16-mm. motion pictures have recently been completed by the Bureau of Public Roads. *Highway Statistics, 1963*, may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for $1 prepaid. Prints of the two films: *Sand Drains—Theory, Design, and Construction* and *Spur Dikes* may be borrowed from the Bureau of Public Roads upon a request submitted directly to the Photographic Section, Bureau of Public Roads, Washington, D.C., 20235. There is no charge for the loan of these films, but express or postage fees must be paid by the borrower. Requests should be submitted well in advance of the desired screening date, and alternate dates indicated, if possible. After each booking, immediate return of the film is required.

Highway Statistics, 1963

Highway Statistics, 1963, a 157-page publication, is the 19th in the annual series that presents statistical and analytical tables of general interest on motor fuel, motor vehicles, highway-user taxation, State and local highway financing, road and street mileage, and Federal aid for highways. Some of the previous annual issues of the series and the summary to 1955 are also available from the Superintendent of Documents; a list of available issues and their purchase prices is carried on the inside back cover of this magazine.

Sand Drains—Theory, Design, Construction

At the request of the Materials Division, Office of Research and Development, a 16-mm. color film that has an optical sound track and a running time of 25 minutes has been produced by Public Roads. The Office of Engineering and Operations cooperated with the Materials Division in planning the film *Sand Drains—Theory, Design, Construction* and approved its technical content. This movie contains a description of sand drains, including the theoretical aspects of their design and construction. The film was shot on a section of Interstate Highway 495 near Alexandria, Va. This movie was filmed primarily for use in the Public Roads engineering training program. It is also expected to be of interest and help to other highway engineers and civil engineering students.

Spur Dikes

Sponsored by the Hydraulics Branch of the Bridge Division in the Office of Engineering and Operations, *Spur Dikes*, a new 16-mm. color film, has an optical sound track and a running time of 15 minutes. Animated artwork, scenes of laboratory models operating under flow conditions, and of an existing spur dike under unusual flood conditions are included in the movie. The scenes of operating models were filmed during research on spur dike models at Colorado State University, sponsored by the Alabama State Highway Department and the Mississippi State Highway Department in cooperation with the Bureau of Public Roads. Also included in the film are scenes, made during a major flood, of the performance of a field installation on the Susquehanna River at Nanticoke, Pa. Plan and cross section views of the shape and size recommended for spur dikes by the Bureau of Public Roads are also included. Use of this movie is expected to be helpful in training highway and bridge engineers in the design and construction requirements for spur dikes.

Moving Picture Technique for Highway Signing Studies— an Investigation of Its Applicability

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BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Establishment of the feasibility of motion picture laboratory tests as a substitute for field research on highway signing was explored in the study reported in this article. Heretofore research related to highway signing generally has been in the form of field tests—a procedure doubly costly in collection of data and research time. Field research is hampered by changing weather conditions, variation in light, interference from traffic, and difficulties associated with obtaining a suitable test site. Development of a suitable laboratory test, it was believed, would eliminate these problems and also permit additional control of variables that might affect the results.

The study reported here was undertaken to validate the substitution of the laboratory tests, in which a moving picture technique was used, for the more commonly used field research. Comparisons of results of field and laboratory tests are reported for the mean legibility distances of signs having two, four, and six legends. The number of errors made by the test participants, the possible effect of their guessing, and the effect of the length of the test word were investigated.

Results of the study established that a laboratory test can be substituted for field tests if the researcher is interested in determining functional relationships between variables. If absolute values are desired, a correction factor must be applied to the laboratory data if they are to reasonably represent field conditions.

Introduction

WHEN construction of the National System of Defense and Interstate Highways was approved in 1956, interest increased in American signing practices. Many prominent engineers expressed the need for signing practices that would keep pace with advances in geometrie highway design.

The American Association of State Highway Officials has published a standard for signing highways in the Interstate System, and the National Joint Committee on Uniform Traffic Control Devices has issued a manual in whieh an entire section is devoted to expressway signing. Many of the design elements incorporated into the signing standards are the results of professional judgment and not research. Although these standards are generally adequate, a need remains for continuing research that will determine the signing requirements for highways and fulfill the needs of motorists.

An extensive research program is required to provide insight into the many eomplex variables involved in the design of highway signs. A program of this magnitude requires that research manpower be used efficiently and economically. It is impossible to determine with certainty what constitutes efficient and economic utilization of researeh manpower, but a brief summary of past and present researeh methods may provide a useful perspeetive.

Research in the field of highway signing generally has been in the form of field studies in which large amounts of data were colleeted at considerable eost and, probably more important, at the expense of considerable time of the researcher. Field variables were usually difficult and sometimes impossible to control. Hindrances to research included the unpredietability of the weather, changes in light conditions, interference from traffic, and lack of a suitable test site at the time it was needed. The result of such postponements is an inefficient use of the researcher's time. Although the foregoing is an oversimplification of the research problem related to signing, it does demonstrate that conducting a field study on signing requirements is not as simple as it might appear to be.

A laboratory study on signing would eliminate weather, light, and traffic problems; reduee site seleetion diffieulties; reduee the time required for obtaining data; and provide the researcher with additional control over the variables. A moving picture technique in whieh a 16-mm. color film would be used appeared to offer the greatest promise for a successful laboratory study. Sueh a teehnique was employed by Burg and Hulbert (*1*)[2] with apparent sueeess in a study made in California.

Some researchers are satisfied that the substitution of a laboratory for a field test will provide representative results, but others are skeptical because validation has not been established. The study reported here was undertaken to validate the substitution; it was performed by comparing results of field and laboratory studies on legibility distances for signs having two, four, and six legends.

Conclusions

The following conclusions have been drawn from the analysis of the study data:

A laboratory test can be substituted for a field test of highway signs if the researcher is interested in determining funetional relationships between variables. But, if absolute values are desired, a correction factor must be applied to the laboratory data if it is to reasonably represent field conditions.

Based on the test information and the analysis of the data, it is concluded that

[1] Condensation of a thesis submitted to the Bureau of Highway Traffic, Yale University, May 1963.

[2] References indicated by italic numbers in parentheses are listed on p. 147.

the motion picture technique offers much promise as a useful research tool in the evaluation of highway signs; and that further efforts at perfecting, validating, and applying the motion picture technique are warranted.

Study Site

A completed but unopened section of Interstate highway readily accessible from Washington, D.C., was selected as the site for the field study. The section is a four-lane, divided, portland cement concrete freeway that has lanes 12 feet wide and asphaltic concrete shoulders 10 feet wide. The design speed is 70 m.p.h., maximum curvature is less than 1 degree, and maximum gradient is less than 3 percent. The site is approximately 1 mile long and is free of sight restrictions.

Experimental Design

The 1-mile advance warning sign (*2*) employed on Interstate highways was used as a guide in designing the test signs. For the 1-mile sign, legends are composed of 16-inch Series E capital letters and 12-inch lowercase letters; the resultant sign has an overall dimension of 15 by 11 feet. Use of a sign of this size would have caused considerable difficulty in sign erection and in message rearrangement. Therefore, ⅓-scale letters were used for the signs in the study reported here. The signs were each 60 inches high and 40 inches wide; the letter heights were 5.3 inches for Series E capital letters and 4 inches for lowercase letters. A vehicle test speed of 20 miles per hour was selected to permit the driver about the same time for reading the sign that he would have had for reading the standard sign when traveling 60 miles per hour.

Hulbert and Burg (*3*) determined that the recognition distance for signs on which word lengths are unequal is longer than for signs on which all the words are of equal length. They also determined that the degree of observer familiarity with the legend is an important factor in identifying a message. To eliminate the effect of these variables from the study discussed here, only six-letter words composed of two, three-letter nonsense syllables were used. The nonsense syllables were pronounceable and had an association value in the range of 27 to 47 percent (*4*). Association value is a measure of the relative ease with which the nonsense syllables suggest words. Any range of association values could have been selected as long as all the test words came from the same range.

Another determinant of legibility distance is the composition of letters that form the word. Neu (*5*) assigned the letter E a legibility value of unity and established the legibility of the other letters of the alphabet in relation to E. The values of legibility that he obtained were used to equalize the relative legibility level of each of the test words selected for this study. The summation of the legibility values of each letter comprising a test word was equal to 6.50 ± 0.01. Neu's article contains a detailed discussion on values of legibility for test words.

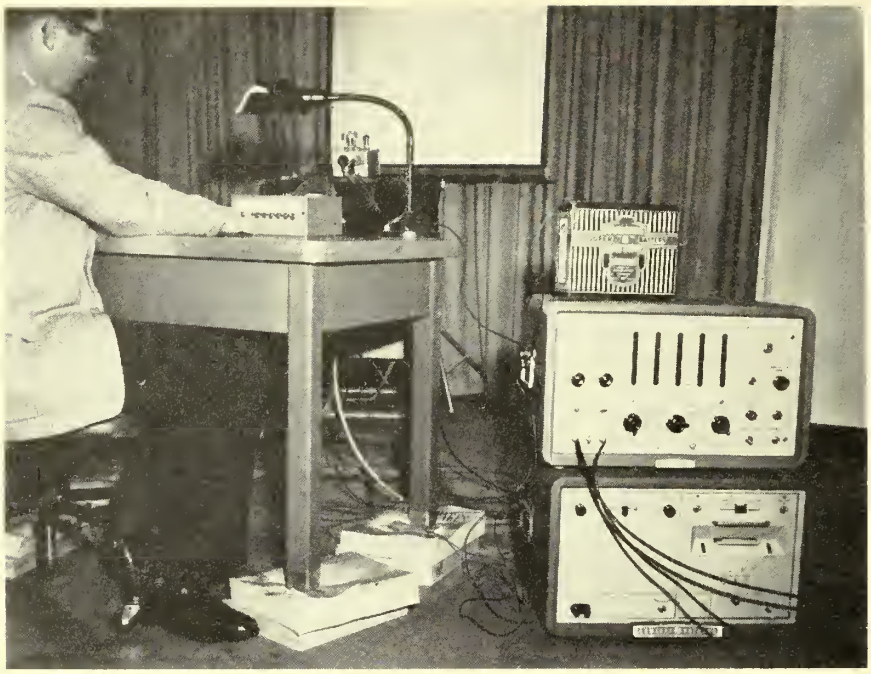

Figure 1.—Test participant operating laboratory instrumentation: keyboard on table at left, digital recorder at lower right below the digital timer.

The position of a word on a sign in relation to the total message is a factor that affects the test participants' ability to locate the word (*6*). A word on the first or last line of a list-type destination sign is located sooner than one in the center of the sign. Therefore, the test words selected for this study, Dahyum and Ruklah, were positioned to balance this effect. The test word was placed in each position on the two- and four-legend signs and in positions one, three, four, and six on the six-legend signs. The test word was not used in positions two and five on the six-legend sign because the increased accuracy that would be obtained did not justify the extra signs needed. Each observer viewed four each of the two-, four-, and six-legend destination signs. An average of these four observations provided an estimate of the observer's legibility distance, which was independent of the word positioning on the sign.

In addition to the 12 signs on which the test words were used, 3 signs that did not have the test words on them were used in the study reported here. These signs, hereafter referred to as blank signs, were used to evaluate the effect of any guessing by the observers. Five other signs were used as an introductory run in which the participants practiced the test procedure. Accordingly, 20 test signs were shown in random order in four runs of a five-sign test course.

All test participants were Junior Engineers in the Bureau of Public Roads Training Program; their ages ranged from 22 to 35 years, and all had average visual acuity (20/20 to 20/40)—some of the engineers had their vision corrected with glasses. The 60 engineers were divided into two equal groups, the first 30 were participants in the field test and the other 30 were assigned to the laboratory test.

Field Test

Test sign locations were spaced 1,000 feet apart and located 10 feet from the right edge of the pavement. Distances along the road from the face of the sign were marked by coded plates to prevent the participant from cuing on the plates by memorizing their order of presentation. This cuing could easily have been accomplished if the plates had been presented in numerical or alphabetical order.

Three cars, each equipped with calibrated speedometers marked for a true 20 miles per hour, were used in the test. Each engineer drove through the course in one of the test cars, an observer was also in the test car. The observer noted and recorded the distance at which the participant read the proper destination and the line on which he indicated that it was located.

A member of the field test group stayed at each sign during the tests to rearrange the messages between test runs. The messages, to be placed on the sign boards, were on individual cardboard plates and could be easily interchanged. Each plate was hung on hooks that had been secured in the proper position before the test was started.

Complete data for each of the two-, four-, and six-legend signs were collected in the field test for 28 test participants.

Laboratory Test

A previously filmed 16-mm. color moving picture of the 20 study signs was used in the laboratory test. The order of presentation

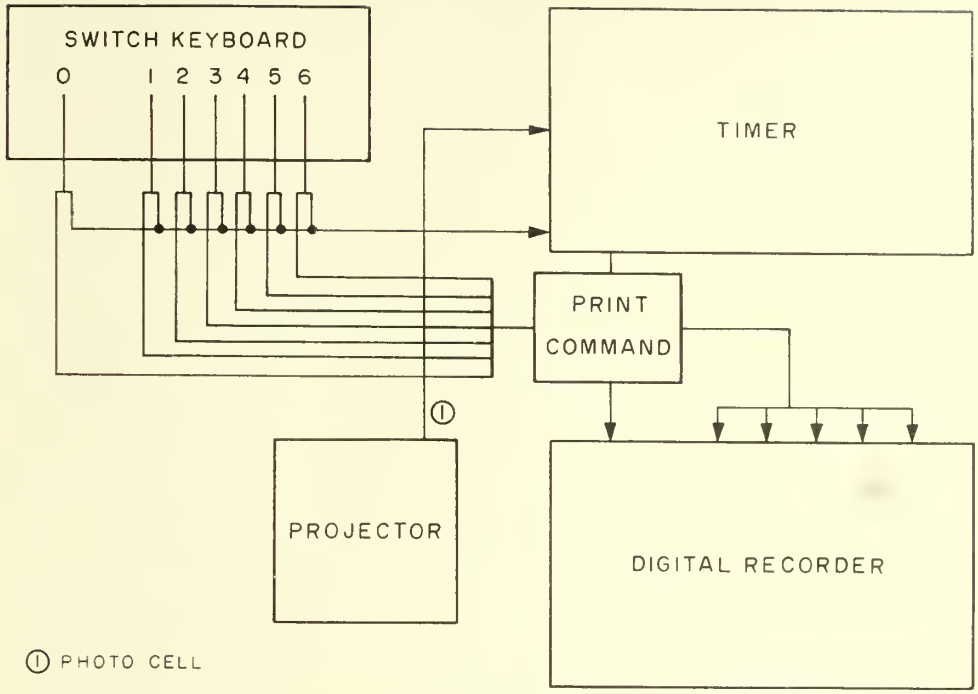

Figure 2.—Block diagram of laboratory test instrumentation.

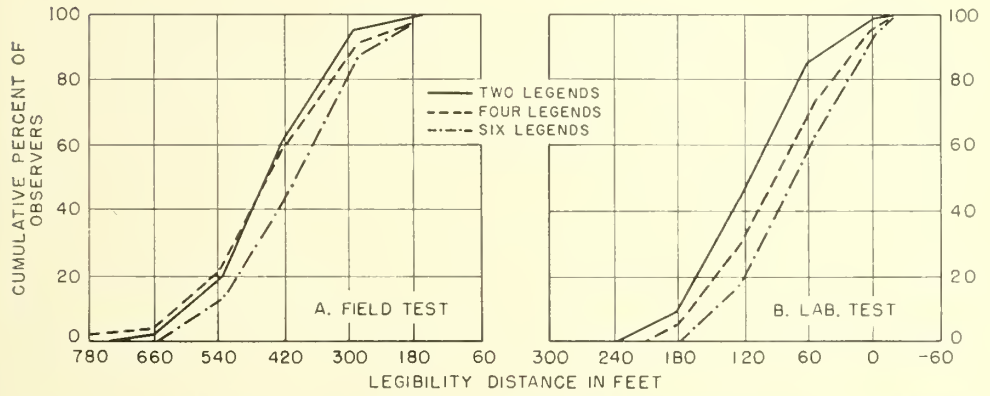

Figure 3.—Cumulative frequency curves for observers legibility distances for the test word Dahyum on signs having two, four, and six legends.

was identical to that employed for the field test. Equipment used in the laboratory tests is shown in figures 1 and 2. The test participant was seated behind a table that had been placed 6 feet in front of and 2 feet to the right of the screen. A control box (switch keyboard) was mounted on the table and cables connected it to an electronic timer and a digital recorder. A timer-actuating box, consisting of a light sensitive photocell and a contact relay, was focused on the screen.

The use of this equipment is best explained by examining the steps involved in a hypothetical test case. Prior to beginning the test, the following instructions were read to the participant: "You will be shown a film, which is composed of 20 highway signs. Each sign will have from two to six destinations listed on it. You will be required to follow an assigned destination throughout the film. This destination may or may not appear on any given sign. You will indicate this by depressing the proper switch, one through six for lines one through six, or zero if your destination does not appear on the signs. You should make your selection as soon as you can read the sign. Your assigned destination is Dahyum."

The 16-mm. color film was then started and at a point 480 frames (approximately 600 feet) in front of the sign, a blank frame, which had previously been inserted, actuated the light sensitive photocell that sent a starting voltage to the timer. When the participant decided on which line his destination was located, he depressed the proper switch—one through six, or zero if the destination were not on the sign. Depression of the switch stopped the timer; the time, to the nearest millisecond, and the number of the switch depressed were automatically recorded on a digital recorder. The film was continued and another sign came into view, whereupon the procedure described was repeated.

Table 1.—Summary of analysis of variance for the differences between 4- and 6-legend signs

	Sum of squares	Degrees of freedom	Mean square
FIELD TEST [1]			
Source of variation:			
Between number of legends	191	1	191
Between observers	434	54	8
LABORATORY TEST [2]			
Source of variation:			
Between number of legends	8	1	8
Between observers	67	50	1.3

[1] The F, ratio for the field test was 23.6, and the significance level was 0.01.
[2] The F, ratio for the laboratory test was 7.3, and the significance level was 0.01.

The elapsed time from the blank frame to the moment the observer closed the switch was subtracted from the total time between the blank frame and the sign to determine the elapsed time between the closing of the switch and the passage of the sign. In the laboratory test, complete data for each of the two-, four-, and six-legend signs were collected for 26 test participants.

Statistical Analysis

A normal distribution curve was fitted to the data by means of a chi-square goodness of fit test. The cumulative curves shown in figure 3 did not significantly depart from a normal distribution (0.05 level). The 0.05 level of significance indicates that there are no more than five chances in a hundred that a fit as good as this could have occurred by chance alone.

Once normality had been established, it was possible to employ the standard deviation to determine the minimum sample size for the desired level of confidence and degree of accuracy. The level of confidence selected for this study was 95 percent and the desired degree of accuracy was 10 percent of the mean observed legibility distance. As the largest standard deviation obtained was used in this determination and the required sample size of 22 participants was less than the 26 and 28 participants used in the laboratory and field tests, respectively, it was concluded that the sample size of observations obtained was sufficient for the level of confidence and degree of accuracy desired.

The sample size being sufficiently large to provide the desired degree of accuracy, the statistical significance of the observed differences between two-, four-, and six-legend signs for the field and laboratory tests remained to be established. The analysis of variance was selected for this determination, as it is a powerful tool for evaluating differences between means (*3*). Further, each test was subdivided and the significance of the differences between two- and four-legend signs were analyzed separately from the four- and six-legend signs. In both the laboratory and

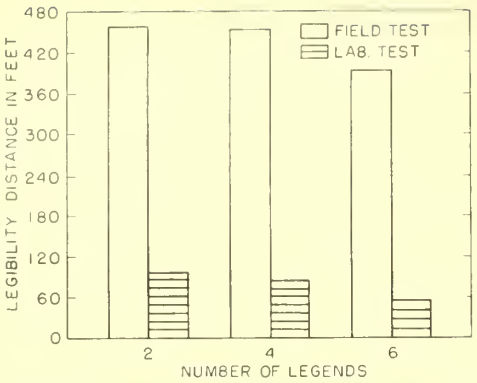

Figure 4.—Mean legibility distances at which participants could accurately identify the test word in field and laboratory tests.

field tests, significant differences in means between four and six legends were noted, but none were found between the two- and four-legend signs. The significant data are summarized in table 1.

Results

The mean legibility values for two-, four-, and six-legend signs were plotted against the number of legends for both the laboratory and field tests as shown in figure 4. The laboratory test results had the same functional relationship as the field test results but a considerable difference existed between the absolute value of legibility distances for the two tests. This difference would not have been a serious problem if the objective had been to determine the maximum desirable number of messages on a sign; both tests had identical results in terms of functional relationships. However, if the objective had been to determine the absolute legibility distance of the sign message, a correction factor would have been necessary to obtain agreement between the results of the two tests. The legibility distances for the field test were approximately five times as great as the corresponding laboratory test results. Better agreement in results could probably have been obtained by improving the quality of the pictures in the film used for the laboratory test.

It was noticed during the test, in spite of all the precautions taken, that the test word Dahyum had characteristics causing it to stand out from the other words. Most serious among these characteristics was its length. The width of the letters and the required letter spacings combined to make this word approximately 6 inches longer than the other words on the sign, as shown in figure 5. This effect was more noticeable in the field test than in the laboratory test and could have contributed to the high ratio obtained between the two tests. The effect of the word length will be discussed in greater detail later.

A comparison of the mean legibility distances obtained in the study reported in this article, 110 and 23 feet per inch of *loop height*, respectively, for the field and laboratory tests, with those obtained in other studies indicates that the field legibility distances are somewhat high and the laboratory legibility distances considerably low. In 1957 the Bureau of Public Roads conducted an extensive sign test in Riverdale, Md., and obtained a mean legibility value of 100 feet per inch of *loop height* for 12-inch, lowercase alphabets. Because the average age of the observers was somewhat greater than the ages of the participants in the study discussed here, the slightly lower legibility value is to be expected.

Figure 5.—The six-legend destination sign, which shows the comparatively longer key word Dahyum.

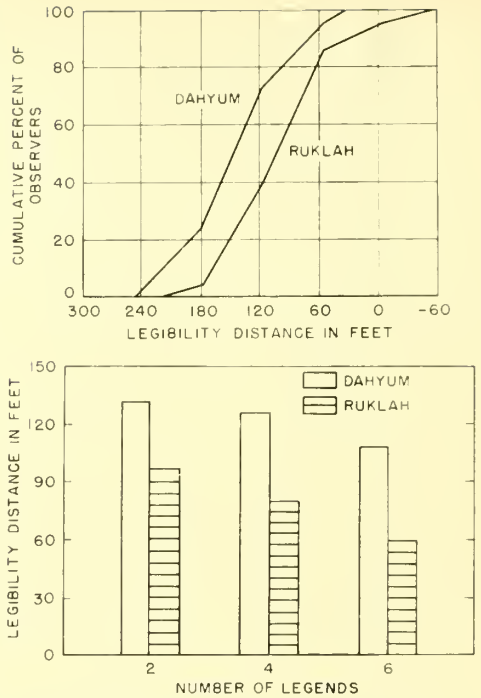

Figure 6.—Comparison of laboratory test results for key words Dahyum (long) and Ruklah (average length). The top part of the figure shows cumulative frequency curves for legibility distances for two-legend signs. The lower part of the figure shows mean legibility distances for two-, four-, and six-legend signs.

Effect of word length

To evaluate the effect of word length, 12 laboratory test participants were asked to locate the word Ruklah as well as the word Dahyum. Ruklah is an average length word but, as previously stated, Dahyum was approximately 6 inches longer than the average word length. A comparison between the mean legibility distances of each word is shown in figure 6. The longer word caused a greater mean legibility distance than the normal length word. However, the relative distances between points for either word were almost identical. The frequency distribution curves further support these findings.

Next, the number of observation errors made by each participant were examined for each word. An error was recorded when the word was indicated to be on a line other that the correct one. The percentage errors for each word are summarized in table 2. For either word, the percentage errors increased with an increase in the number of legends on the sign. However, considerably fewer errors were made for the longer word Dahyum. This was especially true on the six-legend signs for which approximately 42 percent of the Ruklah observations were incorrect.

These findings are in agreement with the general findings made by Hulbert and Burg in 1957 (*3*). They reported that more errors

Table 2.—Laboratory test (Dahyum versus Ruklah) percentage error

Legends	Error for—	
	Ruklah	Dahyum
Number	*Percent*	*Percent*
2	12.9	3.7
4	18.2	6.2
6	42.4	7.8

Table 3.—Mean legibility distances for the test word Dahyum and for the blank signs

Legends	Mean legibility distance for—	
	Dahyum	Blank sign
LABORATORY TEST		
Number	*Feet*	*Feet*
2	94	78
4	85	39
6	56	24
FIELD TEST		
2	436	407
4	432	345
6	396	322

were made on four-legend than on two- and three-legend signs. Also, they determined that the probability of the sign being missed was greater when the messages were all of equal length than when the lengths were unequal.

Blank sign analysis

Blank signs, that is, signs not having the test word present, were introduced into the study to evaluate the effect of guessing by the participants. It was hypothesized that, if the participants guessed, proportionately more errors would be made on the blank signs than on the signs on which the test word appeared. Also, it was believed that a decrease in legibility distance would be obtained if the participants did not guess but spent more time determining whether the test word was on the sign. Examination of the data showed that less than 1 percent of the blank sign observations were in error. This was considerably less than the 3 to 8 percent of errors previously mentioned for the test word Dahyum.

The mean legibility distances for the blank signs were compared with those obtained for Dahyum; the results are tabulated in table 3. The means for the blank sign were consistently less than the means for the test word Dahyum. This may be explained by an increase in the reading time required before the participant could be sure that his test word was not on the sign. Because few errors were made for the blank signs and the resultant means were considerably less than for the test word, it appears that under the terms of the hypothesis, guessing did not materially affect the results of the study.

Summary of Findings

For both the field and laboratory tests, a decrease in legibility distance was obtained when the number of legends on the sign was increased from four to six. No differences were obtained when the number of legends was increased from two to four.

The number of observation errors made by the participant increased as the number of legends was increased for both test conditions.

In the laboratory test, a test word destination longer than all the other words on the sign, which were equal in length, produced a substantial reduction in the mean observation distance and the number of participants' observation errors.

Guessing by participants did not materially affect the results of this study.

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Motor-Vehicle Travel Estimates

1963 United States 1962 State-by-State

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

By THEODORE S. DICKERSON, JR., and IVANO E. CORVI, Highway Engineers, Current Planning Division

MOTOR-VEHICLE travel in the United States in 1963 totaled 801.1 billion vehicle-miles, an increase of 4.5 percent over the travel in 1962. The travel data were compiled by the Bureau of Public Roads from information supplied by the State highway departments and toll authorities. Total travel for 1964, based on information for the first 10 months of the year, is estimated at 838 billion vehicle-miles, a 4.6-percent increase over 1963.

The proportions of travel by road system and vehicle type changed little from 1962 to 1963. Of the 1963 travel, 37.6 percent was on main rural roads comprising 14 percent of the Nation's total of 3.6 million miles of roads and streets. Some 48.0 percent of the travel was on urban streets, which comprise only 13 percent of the total mileage. Travel on local rural roads was only 14.4 percent, although these roads are 73 percent of the total mileage.

Passenger cars represented 84 percent of the vehicles registered and did 82 percent of the travel in 1963: trucks and truck combinations accounted for 16 percent of the vehicles and 18 percent of the travel; similar figures for buses were less than 1 percent.

Average vehicle performance in 1963 differed very little from that reported for 1962. The average motor vehicle traveled 9,590 miles in 1963, almost half of it in cities, and consumed 760 gallons of fuel at a rate of 12.42 miles per gallon. The average passenger car traveled 9,378 miles and consumed 652 gallons of fuel at a rate of 14.37 miles per gallon. In 1963, the average truck traveled a little more and the average commercial bus less than in 1962, but their average rates of fuel consumption did not change appreciably.

The travel and related information for 1963 and revised information for 1962 are shown

Table 1.—Estimated motor-vehicle travel in the United States and related data, calendar year 1963 and revised 1962 [1]

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total	Average per vehicle	
CALENDAR YEAR 1963										
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gallon*
Passenger cars [2]	234,641	90,368	325,009	329,690	654,699	69,813	9,378	45,545	652	14.37
Buses:										
Commercial	877	170	1,047	1,794	2,841	[3] 82.2	34,562	606	7,372	4.69
School and nonrevenue	642	708	1,350	292	1,642	215.7	7,612	232	1,076	7.08
ALL BUSES	1,519	878	2,397	2,086	4,483	297.9	15,049	838	2,813	5.35
All passenger vehicles	236,160	91,246	327,406	331,776	659,182	70,111	9,402	46,383	662	14.21
Trucks and combinations	64,794	24,270	89,064	52,809	141,873	13,423	10,569	18,133	1,351	7.82
ALL MOTOR VEHICLES	300,954	115,516	416,470	384,585	801,055	83,534	9,590	64,516	760	12.42
CALENDAR YEAR 1962 REVISED										
Passenger cars [4]	224,699	87,573	312,272	316,825	629,097	66,638	9,441	43,771	657	14.37
Buses:										
Commercial	863	167	1,030	1,826	2,856	[3] 78.3	36,475	610	7,791	4.68
School and nonrevenue	630	694	1,324	286	1,610	209.7	7,678	227	1,082	7.09
ALL BUSES	1,493	861	2,354	2,112	4,466	288.0	15,507	837	2,906	5.34
All passenger vehicles	226,192	88,434	314,626	318,937	633,563	66,926	9,467	44,608	667	14.20
Trucks and combinations	61,130	23,007	84,137	49,152	133,289	12,809	10,406	17,089	1,334	7.80
ALL MOTOR VEHICLES	287,322	111,441	398,763	368,089	766,852	79,735	9,618	61,697	774	12.43

[1] For the 50 States and District of Columbia.
[2] Includes taxicabs; also 786,318 motorcycles, which are estimated to account for 0.3 percent of the total travel.
[3] Adjusted to reflect revised procedures for eliminating duplicate motorbus registrations.
[4] Includes taxicabs; also 660,400 motorcycles, which are estimated to account for 0.3 percent of the total travel.

Table 2.—Vehicle-miles of travel in 1962 on all roads and streets, free and toll, by State and highway system, estimated by State highway departments in 1961

(Millions)

DIVISION	STATE	FEDERAL-AID HIGHWAY SYSTEM														NOT ON FEDERAL-AID SYSTEMS						
		INTERSTATE RURAL			INTERSTATE URBAN			SUB-TOTAL INTER-STATE	OTHER FAP RURAL	OTHER FAP URBAN	FAS STATE RURAL	FAS STATE URBAN	FAS LOCAL RURAL	FAS LOCAL URBAN	TOTAL FEDERAL-AID RURAL	TOTAL FEDERAL-AID URBAN	TOTAL FEDERAL-AID	OTHER STATE RURAL	OTHER STATE MUNICI-PAL	LOCAL RURAL	LOCAL MUNICI-PAL	SUB-TOTAL RURAL	SUB-TOTAL URBAN AND MUNICI-PAL	TOTAL
		FINAL	TRAVELED	TOTAL RURAL	FINAL	TRAVELED	TOTAL URBAN																	
NEW ENGLAND	Connecticut	515	85	600	1,145	180	1,325	1,925	1,255	880	950	475	30	65	2,835	2,745	5,580	1,215	1,025	750	2,530	4,800	6,300	11,100
	Maine	223	331	554	42	57	99	653	1,121	339	723	92	-	-	2,398	530	2,928	662	212	376	156	3,436	898	4,334
	Massachusetts	713	316	1,029	458	598	1,056	2,085	2,539	2,823	600	405	1,131	1,104	5,299	5,388	10,687	108	93	3,054	5,227	8,461	10,708	19,169
	New Hampshire	152	104	256	20	53	73	329	903	283	595	74	4	4	1,758	434	2,192	212	50	250	383	2,220	867	3,087
	Rhode Island	35	70	105	71	212	283	388	430	680	150	136	20	155	705	1,254	1,959	70	88	23	1,102	798	2,444	3,242
	Vermont	49	257	306	4	52	56	372	579	95	221	7	102	20	1,208	189	1,396	4	5	181	189	1,393	382	1,775
	TOTAL	1,687	1,163	2,850	1,740	1,162	2,902	5,752	6,827	5,100	3,230	1,189	1,287	1,348	14,203	10,539	24,742	2,271	1,473	4,634	9,587	21,108	21,599	42,707
MIDDLE ATLANTIC	New Jersey	99	698	797	880	1,464	2,344	3,141	2,527	4,008	45	48	1,355	1,258	4,724	7,658	12,382	1,302	1,092	5,684	7,850	11,710	16,600	28,310
	New York	1,864	521	2,385	3,153	915	4,068	6,453	8,437	8,286	1,749	630	2,818	1,359	15,389	14,343	29,732	232	59	6,484	13,887	22,105	28,289	50,394
	Pennsylvania	2,060	1,271	3,331	1,375	1,394	2,769	6,100	8,458	5,331	6,133	3,246	27	63	17,949	11,409	29,358	3,308	2,816	1,752	5,766	23,009	19,991	43,000
	TOTAL	4,023	2,490	6,513	5,408	3,773	9,181	15,694	19,422	17,625	7,927	3,924	4,200	2,680	38,062	33,410	71,472	4,842	3,967	13,920	27,503	56,824	64,880	121,704
SOUTH ATLANTIC (NORTH)	Delaware	-	117	117	42	214	256	373	739	459	250	160	-	-	1,106	875	1,981	-	-	55	63	1,161	938	2,099
	Dist. of Col	-	-	-	37	220	257	257	-	845	-	-	-	459	-	1,571	1,571	-	-	-	866	-	2,437	2,437
	Maryland	204	674	878	304	853	1,157	2,035	2,339	1,521	1,289	657	375	298	4,881	3,633	8,514	355	140	904	3,236	6,140	7,009	13,149
	Virginia	384	2,333	2,717	241	772	1,013	3,730	4,483	1,520	2,275	302	978	120	10,453	2,955	13,408	67	234	1,952	2,059	12,472	5,248	17,720
	West Virginia	152	516	668	11	342	353	1,021	1,589	366	1,127	92	735	26	4,119	837	4,956	3	20	243	1,178	4,365	2,035	6,400
	TOTAL	740	3,640	4,380	635	2,401	3,036	7,416	9,150	4,711	4,941	1,211	2,088	913	20,559	9,871	30,430	425	394	3,154	7,402	24,138	17,667	41,805
SOUTH ATLANTIC (SOUTH)	Florida	380	1,597	1,977	351	1,117	1,468	3,445	4,332	1,635	3,783	1,177	603	129	10,695	4,409	15,104	722	957	1,282	5,795	12,699	11,161	23,860
	Georgia	284	1,423	1,707	325	667	992	2,699	4,626	1,195	2,159	395	1,035	264	9,527	2,846	12,373	164	307	1,585	3,457	11,276	6,610	17,886
	North Carolina	624	989	1,613	121	158	279	1,892	3,462	1,051	4,434	505	3,322	663	12,831	2,498	15,329	52	113	865	2,417	13,748	5,028	18,776
	South Carolina	402	723	1,125	5	226	231	1,356	3,037	887	2,897	281	88	3	7,147	1,402	8,549	222	610	361	140	7,730	2,152	9,882
	TOTAL	1,690	4,732	6,422	802	2,168	2,970	9,392	15,457	4,768	13,273	2,358	5,048	1,059	40,200	11,155	51,355	1,160	1,987	4,093	11,809	45,453	24,951	70,404
EAST NORTH CENTRAL	Illinois	1,029	1,643	2,672	2,728	1,108	3,836	6,508	7,582	5,573	900	353	1,754	141	12,908	9,903	22,811	1,481	3,256	2,583	9,629	16,972	22,788	39,760
	Indiana	829	1,517	2,346	192	823	1,015	3,361	5,040	2,055	2,601	328	1,317	72	11,304	3,470	14,774	73	16	828	6,495	12,205	9,981	22,186
	Michigan	2,040	983	3,023	851	1,334	2,185	5,208	6,190	3,228	1,043	175	5,320	666	15,576	6,254	21,830	14	43	3,722	8,889	19,312	15,186	34,498
	Ohio	2,048	1,229	3,277	752	2,241	2,993	6,270	7,169	4,599	3,735	894	1,735	870	15,916	9,456	25,372	165	647	4,780	9,119	20,861	19,222	40,083
	Wisconsin	532	472	1,004	48	249	297	1,301	4,741	1,487	1,450	300	1,315	730	8,510	2,814	11,324	6	52	1,003	4,480	9,519	7,346	16,865
	TOTAL	6,478	5,844	12,322	4,571	5,755	10,326	22,648	30,722	17,042	9,729	2,050	11,441	2,479	64,214	31,897	96,111	1,739	4,014	12,916	38,612	78,869	74,523	153,392
WEST NORTH CENTRAL	Iowa	260	486	746	55	144	199	945	4,863	1,122	-	-	1,386	48	6,995	1,369	8,364	50	28	823	2,422	7,868	3,819	11,687
	Kansas	408	334	742	172	164	336	1,078	3,471	935	524	16	1,164	255	5,901	1,542	7,443	28	29	645	3,195	6,574	4,766	11,340
	Minnesota	66	778	844	186	817	1,003	1,847	4,250	1,805	897	26	2,179	125	8,170	2,960	11,130	4	117	1,036	2,933	9,210	6,010	15,220
	Missouri	530	1,422	1,952	586	647	1,233	3,185	4,270	1,491	1,776	333	29	36	8,027	3,093	11,120	89	445	1,852	5,669	9,968	9,208	19,176
	Nebraska	134	489	623	23	70	93	716	2,282	433	469	22	571	37	3,945	585	4,530	-	-	952	1,688	4,897	2,273	7,170
	North Dakota	132	272	404	4	52	56	460	1,121	78	164	2	429	13	2,118	149	2,267	-	-	607	353	2,725	502	3,227
	South Dakota	140	384	524	7	10	17	541	1,432	178	158	21	390	19	2,504	235	2,739	10	10	565	307	3,079	552	3,631
	TOTAL	1,670	4,165	5,835	1,033	1,904	2,937	8,772	21,589	6,043	3,988	420	6,148	533	37,660	9,933	47,593	181	630	6,480	16,567	44,321	27,130	71,451
EAST SOUTH CENTRAL	Alabama	206	1,099	1,305	17	493	510	1,815	3,140	1,239	1,013	207	1,673	174	7,131	2,130	9,261	34	43	420	2,619	7,585	4,792	12,377
	Kentucky	260	862	1,122	112	514	626	1,748	3,125	674	2,614	209	444	88	7,305	1,597	8,902	453	414	902	1,560	8,660	3,571	12,231
	Mississippi	138	680	818	54	370	424	1,242	2,870	527	640	64	744	105	5,072	1,120	6,192	5	21	520	955	5,597	2,096	7,693
	Tennessee	97	1,713	1,810	202	626	828	2,638	3,651	1,199	966	577	547	54	6,974	2,658	9,632	31	3	1,545	2,963	8,550	5,624	14,174
	TOTAL	701	4,354	5,055	385	2,003	2,388	7,443	12,786	3,639	5,283	1,057	3,408	421	26,482	7,505	33,987	523	481	3,387	8,097	30,392	16,083	46,475
WEST SOUTH CENTRAL	Arkansas	105	746	851	21	277	298	1,149	2,302	485	1,661	250	265	31	5,110	1,064	6,174	17	57	772	870	5,899	1,991	7,890
	Louisiana	109	1,117	1,226	57	688	745	1,971	1,918	821	2,620	390	3	-	5,767	1,956	7,723	481	256	320	994	6,568	3,206	9,774
	Oklahoma	716	540	1,255	220	542	762	2,018	5,052	1,171	1,179	246	629	157	8,116	2,336	10,452	85	180	949	515	9,150	3,031	12,181
	Texas	835	2,682	3,517	2,219	2,322	4,541	8,058	8,298	5,022	5,710	1,897	-	-	17,525	11,450	28,985	1,143	462	2,254	11,743	20,922	23,665	44,587
	TOTAL	1,765	5,085	6,850	2,517	3,829	6,346	13,196	17,570	7,499	11,170	2,783	928	188	36,518	16,816	53,334	1,726	955	4,295	14,122	42,539	31,893	74,432
MOUNTAIN	Arizona	395	1,396	1,791	91	322	413	2,204	850	111	505	88	537	1,220	3,773	1,832	5,605	128	19	667	966	4,568	2,817	7,385
	Colorado	344	786	1,130	127	483	610	1,740	1,882	903	820	105	-	-	3,832	1,618	5,450	31	38	1,093	1,909	4,956	3,565	8,521
	Idaho	152	523	685	10	78	88	773	1,254	119	358	16	249	21	2,546	244	2,790	2	4	555	485	3,103	733	3,836
	Montana	112	588	700	1	89	90	790	1,204	164	332	28	-	-	2,236	282	2,518	-	-	712	444	2,948	726	3,674
	Nevada	168	480	648	-	84	84	732	550	144	192	23	56	30	1,406	281	1,777	27	36	93	153	1,616	460	2,076
	New Mexico	325	865	1,190	21	236	257	1,447	1,313	387	700	111	56	1	3,215	756	3,971	56	315	316	582	3,627	1,653	5,280
	Utah	98	889	987	6	488	494	1,481	580	261	560	219	159	28	2,386	1,002	3,388	24	121	337	523	2,747	1,646	4,393
	Wyoming	140	561	701	9	59	68	769	730	55	213	1	16	12	1,660	136	1,796	2	3	246	352	1,908	491	2,399
	TOTAL	1,744	6,088	7,832	265	1,839	2,104	9,936	8,473	2,144	3,770	631	1,063	1,312	21,144	5,151	27,295	310	526	4,019	5,414	25,473	12,091	37,564
PACIFIC	California	549	5,040	5,589	4,235	4,762	8,997	14,586	9,014	9,502	1,884	910	3,648	3,078	19,135	22,587	41,722	495	1,200	12,035	25,828	31,665	49,624	81,289
	Oregon	709	559	1,268	232	120	352	1,620	2,022	806	932	123	732	105	4,954	1,386	6,340	57	29	1,044	1,483	6,055	2,898	8,953
	Washington	310	1,447	1,757	221	971	1,192	2,949	2,355	530	1,248	185	1,227	250	6,583	2,257	8,845	54	98	2,584	2,224	9,226	4,579	13,805
	TOTAL	1,568	7,045	8,614	4,688	5,853	10,541	19,155	13,392	11,038	4,064	1,218	5,607	3,433	30,677	26,230	56,907	606	1,336	15,663	29,535	46,946	57,101	104,047
TOTAL - ALL DIVISIONS		22,066	44,607	66,673	22,044	30,587	52,631	119,404	154,444	79,909	67,334	16,801	41,218	14,366	329,719	163,507	493,226	13,783	15,763	72,561	168,648	416,063	347,918	763,981
	Alaska [1]	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	Hawaii	-	225	225	-	149	149	374	457	122	186	11	-	-	868	282	1,150	-	-	317	607	1,185	889	2,074
UNITED STATES TOTAL		22,066	44,832	66,898	22,044	30,736	52,780	119,778	154,901	79,731	67,520	16,812	41,218	14,366	330,587	163,789	494,376	13,783	15,763	72,878	169,255	417,248	348,807	766,055

[1] Alaska has no interstate mileage and therefore did not prepare an estimate. Total 1962 vehicle-miles in Alaska are estimated to be 797 million.

in table 1 by road system and vehicle type. Such data have been reported in PUBLIC ROADS magazine for a number of years; the latest for 1962 appeared in volume 32, No. 11, December 1963, p. 268.

Each State highway department prepared 1962 travel estimates on a uniform basis as a foundation for Traffic forecasts needed for the report to The Congress, *1965 Estimate of the Cost of Completing the National System of Interstate and Defense Highways.* These estimates, on a uniform basis, were prepared in accordance with instructions issued by the Bureau of Public Roads. Detailed State-by-State travel estimates by highway system are shown in table 2.

According to the State estimates, the traveled way of the Interstate System carried 119.8 billion vehicle-miles, or 15.6 percent of the total 1962 travel on all roads and streets. The traveled way consisted of 11,000 miles of Interstate System highways now in use and 30,000 miles of existing connecting highways; service for this total mileage will be provided by the completed Interstate System. From the State estimates it is expected that by 1975 the 41,000-mile Interstate System, comprising little more than 1 percent of the total road and street mileage of the United States, will carry 285 billion vehicle-miles, or nearly 25 percent, of the total 1,165 billion miles of travel estimated for 1975. Earlier estimates had indicated that the Interstate System would be carrying more than 20 percent of all traffic in 1971.

According to the State estimates of 1962 travel, all Federal-aid systems combined, which includes about 24 percent of all roads and streets, carried 65 percent of all travel. Because of their principal use in a report to The Congress, the State estimates of 1962 travel were made according to a system classification and rural-urban distinction directly related to the Federal-aid program. In the Federal-aid law, an urban area is "an area including and adjacent to a municipality or other urban place having a population of 5,000 or more . . . ." In the annual estimates reported in table 1, however, urban signifies the areas within the political boundaries of municipalities such as cities, boroughs, and villages. As a consequence, urban travel in 1962 as shown in table 1 was 48.0 percent of the total, but by the State estimates it was 45.5 percent.

In recent years the annual travel estimates reported in table 1 have been developed in part from trend indicators, extending from a base of comprehensive studies by the States of travel in 1957. The State estimates of travel in 1962 have now provided a new base, and from this base the revised estimates for 1962, also shown in table 1, were prepared. The revised estimate of total travel in 1962 is only 0.1 percent less than the original estimate. One of the largest differences was in the proportion of urban travel, which was revised from 46 percent of total travel to 48 percent.

Films Available from the Bureau of Public Roads

In addition to the two most recent films—*Sand Drains* and *Spur Dikes*—described herein under *New Publications and Films*, the following described films produced by the Bureau of Public Roads are available to Government agencies, State and local highway departments, engineering colleges and universities, and other responsible organizations interested in highway development and improvement. No charge is made for the use of these films but the borrower must pay transportation charges both ways. Requests for films should be submitted well in advance of the desired showing and alternate dates for showing should be given. The films must be returned promptly. Correspondence concerning these films should be addressed to: Photographic Section, Bureau of Public Roads, Washington, D.C., 20235.

THE PENNSYLVANIA PILOT STUDY: 16-mm. color sound—25 minutes (1949). A technical movie filmed on the Pennsylvania Turnpike and parallel route to illustrate the performance of heavy motor vehicles as affected by varying conditions of load, road, and grade. The tests were conducted by the Bureau of Public Roads in cooperation with the Committee on Economics of Motor Vehicles Size and Weight of the Highway Research Board and other allied organizations.

HIGHWAY SOIL ENGINEERING: 16-mm. color sound—1 hour 50 minutes (1950). A technical film illustrating the methods employed for surveying and sampling in the field and testing in the laboratory the subgrade soils encountered in highway construction. The tests are those practiced by the Bureau of Public Roads and many State highway departments in accordance with standards adopted by the American Association of State Highway Officials.

INTER-AMERICAN HIGHWAY REPORT—PART I, MEXICO: 16-mm. color sound—55 minutes (1951). The film illustrates the present conditions of the Inter-American Highway in Mexico and shows what the traveler may expect with respect to scenery, highway conditions, and accommodations. Animated maps portray the location of the highway south from the American border at Laredo, Tex., to the junction with the Inter-American Highway in Guatemala.

INTER-AMERICAN HIGHWAY REPORT—PART II, CENTRAL AMERICA: 16-mm. color sound—62 minutes (1952). The film illustrates the present conditions of the southerly 1,600 miles of the Inter-American Highway extending from the Mexico-Guatemala boundary through the Central America Republics of Guatemala, El Salvador, Honduras, Nicaragua, and Costa Rica, and hence through Panama to Panama City. Animated maps show the location of the highway in each country and define the passable sections.

ROAD TEST ONE, MD.: 16-mm. color sound—78 minutes (1953). This film depicts the operation of and explains the conclusions drawn from the test conducted in 1950 at La Plata, Md., under the direction of the Highway Research Board in behalf of 11 Eastern States and with the cooperation of the Bureau of Public Roads, to study the effects of controlled truck traffic of different loadings on a concrete pavement. The nature and causes of pumping, cracking, and stresses are demonstrated by means of working models.

PRECAST CONCRETE BRIDGE: 16-mm. color sound—18 minutes (1955). The film illustrates, with scenes at an actual bridge site and with animated drawings, a method of bridge building that is growing in usage in this country. The nature of the forms for the precast concrete beams, deck slabs, and curb sections, and their use in the central casting yard, are demonstrated in detail. Driving of piles and construction of bent caps, placement of the precast units, and the final operations involved in completing the three-span structure are shown step by step.

LOST PRODUCTION IN HIGHWAY CONSTRUCTION: 16-mm. color sound—30 minutes (1957). The film examines minor delays that affect production rates of key units of highway construction equipment including power shovels, scrapers, hot-mix bituminous plants, and concrete paver. Contrasts in operation practices shown in the motion picture are very revealing.

POWER SHOVEL PRODUCTIVITY: 16-mm. color sound—30 minutes (1958). The film, based on extensive studies conducted by Public Roads highlights the job conditions that determine the yardage output of power shovels on highway grading work and demonstrates how production is affected by the speed of dipper cycle, size of dipper load, and frequency and duration of minor delays.

LOST MIXING TIME ON DUAL-DRUM PAVER: 16-mm. color sound—30 minutes (1959). The film highlights the importance of the simultaneous mixing interval in meeting mixing time specifications with dual-drum pavers. It shows some trouble spots and the significance of proper adjustments to the batchmeter.

DUAL-DRUM PAVER PRODUCTIVITY: 16-mm. color sound—30 minutes (1960). Action scenes from actual paver operations illustrate how and to what extent operating delays affect paver productivity on the jobs where paver production rates ranged from low on some to outstanding on others. The film shows the extent to which the number of batch trucks required to supply the paver is affected by batch plant efficiency and capacity as well as haul road variables. The simultaneous showing of contrasting action scenes effectively highlights fast versus mediocre batch truck dumping performance at the skip.

INTRODUCTION TO HIGHWAY HYDRAULICS: 16-mm. color sound—21 minutes (1960). This film illustrates some fundamental principles of hydraulics of open channel flow. The principles are illustrated by hydraulic models in the laboratory and correlated with field highway installations where possible. The film was produced in cooperation with Colorado State University.

WINTER DRIVING: 16-mm. color sound—24 minutes (1960). Produced in cooperation with the National Safety Council, it illustrates safe winter driving techniques for starting, stopping, and cornering on snow- and ice-covered roads, and includes a demonstration of a relatively easy method of installing tire chains. It presents tips on how to avoid skids and shows how to recover from a skid if it does occur. Suggestions on preparing your car for winter driving are also included. This film is especially appropriate for driver education classes, off-the-job safety programs, and civic gatherings and is available to Federal agencies and State highway departments through Bureau of Public Roads. For others, it is available either on a rental basis or by purchase from the National Safety Council, 425 North Michigan Avenue, Chicago, Ill., 60611.

WINTER DRIVING (TV): 16-mm. black and white sound—15 minutes (1962). A shortened version of the film *Winter Driving* produced in cooperation with the National Safety Council and released in 1960. The TV version covers the same information, using the same photography reproduced in black and white. For TV use especially.

SAFETY IN HIGHWAY SURVEYING: 16-mm. color sound—25 minutes (1960). The functions of a highway survey party are shown with the hazards and appropriate precautions for working in rough country, dense vegetation, timberland and mountainous country as well as along the highway. Different hand tools and equipment such as axes, machetes, chain saws, and lifelines are demonstrated together with an explanation of hazards and safe working practices. Survey work and inspections performed in cooperation with actual construction are also included

with emphasis on the hazards peculiar to this phase. Precautions taken for the safety and convenience of the traveling public during survey and construction are shown.

RIGHT-OF-WAY FOR HIGHWAYS: 16-mm. color sound—26 minutes (1961). Shows how a State highway department studies, evaluates, and selects the route for a new highway; the various steps in the appraisal of a property needed for right-of-way; and the negotiation for purchase of the property. The participation of Public Roads in Federal-aid projects is also shown. The film was produced in cooperation with the States of Iowa, Minnesota, Missouri, and Nebraska. Can be used for orientation of new highway personnel, for introductory training in right-of-way work, at public hearings, and for public information generally.

THE WASHO ROAD TEST: 16-mm, color sound—35 minutes (1956). This film depicts the operation and major findings of the large-scale road test undertaken cooperatively by the Western Association of State Highway Officials, the Bureau of Public Roads, and motor vehicle and petroleum industries, under the direction of the Highway Research Board. The road near Malad, Idaho, was especially built to determine the effect of controlled heavy traffic on bituminous pavement.

LAREDO TO PANAMA—THE INTER-AMERICAN HIGHWAY: 16-mm, color sound—18 minutes (1964). The story of the economic and social benefits brought to the people of Mexico and Central America by the Inter-American Highway. Shows the industrial, agricultural, and educational growth developing in these countries and how the highway has been instrumental in improving communications and the exchange of cultures in these ancient lands.

HIGHWAY BRIDGE RESEARCH: 16-mm, color sound—18½ minutes (1964). Shows methods used in conducting strain research on steel and reinforced concrete bridges. Details installation of gages, wiring, and instrumentation; shows operation; explains equipment; and gives results.

THE ROAD TO PROSPERITY: 16-mm, color sound—13½ minutes (1964). Explains the economic impact on America of the Interstate Highway System through on-the-spot statements by four Kansas businessmen and scenes taken over the country.

TARGET: URBAN MOBILITY: 16-mm, color, sound—18 minutes (1965). After describing the rapid population growth taking place in the United States, the film explains the meaning of section 9 of the Federal-Aid Highway Act of 1962 concerning requirements for an urban transportation planning process in urban areas having a population greater than 50,000. It then briefly outlines the 10 basic technical elements for which inventories and analyses are necessary in organizing and carrying out the cooperative, comprehensive, and continuing urban transportation planning process.

HURRICANE HITS THE HIGHWAYS: 16-mm, color sound—13½ minutes (1962). Photography during and after the 1961 hurricane Carla, that struck the Texas gulf coast area, shows the power of the storm and the damage to buildings, highways, and bridges. The film tells the story of the havoc brought by the winds and water, and the role played by Public Roads in cooperation with the Texas Highway Department and other government agencies in rebuilding and repairing roads and bridges so that normal activity could be restored.

AASHO ROAD TEST: CONSTRUCTION AND MATERIALS: 16-mm, color sound—27 minutes (1962). This film follows the history of the construction and illustrates the methods of preparing the test roads for the $27 million AASHO Road Test. It covers the construction aspects from the clearing of the test site in 1956 to completion of the test loops in 1958. It records the quality and control features that were built into each tangent of rigid and flexible pavement and explains the objectives of the research program, "to relate the performance of the pavement and bridges directly to design and loading."

AASHO ROAD TEST: PAVEMENT RESEARCH. 16-mm, color sound 37 minutes (1962). This film covers the objectives, methods, and principal results of the pavement research conducted in the AASHO Road Test. It shows how the pavement loops were tested under specific loads, at known speeds, how the roads stood up to the tests, and the methods used to measure serviceability. The results of the test program are presented dramatically and graphically.

AASHO ROAD TEST: THE ROAD TO BETTER ROADS. 16-mm, color sound—14 minutes (1963). This film explains, in lay terms, the kind of research done in the area of highway construction, as shown by the AASHO Road Test. The picture begins by establishing the need for such projects and the variety of programs involved. The main story is of the construction and operation of the AASHO tests and the accumulation and processing of data.

SKID CORRELATION STUDY: 16-mm, color sound—14 minutes (1963). Compares various techniques used in testing the coefficients of friction on five specially constructed pavements at Tappahannock, Va. Study was for the purpose of standardizing techniques and equipment used by highway departments and research organizations.

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PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–32 are available upon request addressed to Bureau of Public Roads, Washington, D.C., 20235.*

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Annual Reports of the Bureau of Public Roads:

1960, 35 cents. 1963, 35 cents. 1964, 35 cents. (Other years are now out of print.)

REPORTS TO CONGRESS

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways, 1937–61. $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

Calibrating and Testing a Gravity Model With a Small Computer (1964). $2.50.

Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

Federal Laws, Regulations, and Other Material Relating to Highways (1960). $1.00.

Financing of Highways by Counties and Local Rural Governments: 1942–51 (1955). 75 cents.

Highway Bond Financing . . . An Analysis, 1950–1962. 35 cents.

Highway Finance 1921–1962 (a statistical review by the Office of Planning, Highway Statistics Division). 15 cents.

Highway Planning Map Manual. $1.00.

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

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Highway Research and Development Studies Using Federal-Aid Research and Planning Funds (1964), $1.00.

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Highway Statistics, Summary to 1955. $1.00.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Highways and Social and Economic Changes (1964). $1.25.

Hydraulics of Bridge Waterways (1960). 40 cents.

Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). 40 cents. Appendix, 70 cents.

Interstate System Route Log and Finder List. 10 cents.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.

Part V—Traffic Controls for Highway Construction and Maintenance Operations (1963). 25 cents.

Opportunities for Young Engineers in the Bureau of Public Roads (1964) 15 cents.

Peak Rates of Runoff From Small Watersheds (1961). 30 cents.

Reinforced Concrete Pipe Culverts—Criteria for Structural Design and Installation (1963). 30 cents.

Road-User and Property Taxes on Selected Motor Vehicles, 1964. 45 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1958: a reference guide outline. 75 cents.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP–61 (1961). $2.25.

Standard Plans for Highway Bridges (1962):

Vol. I—Concrete Superstructures. $1.00.
Vol. II—Structural Steel Superstructures. $1.00.
Vol. III—Timber Bridges. $1.00.
Vol. IV—Typical Continuous Bridges. $1.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Aerial Surveys in Highway Engineering (1960). 40 cents.

Traffic Assignment Manual. $1.50.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Transition Curves for Highways (1940). $1.75.

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IN THIS ISSUE: *Highway Interchange Area Development*

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IN THIS ISSUE

Highway Interchange Area Development, *by F. I. Thiel* 153

Aerial Color Photography and its Use in Materials Surveys, *by J. R. Chaves and R. L. Schuster* 167

Effectiveness of Sign Background Reflectorization, *by L. D. Powers* 172

Shear Loads on Pavements, *by E. S. Barber* . . 179

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Highway Interchange Area Development

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

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A synthesis of reports on studies defining the problem of land use development of areas near interchanges, the approaches to controlled-access highways—development that often causes traffic congestion on the interchanges—is presented in this article. The location of an interchange can be the stimulus for nearby economic development; but this development can be the source of problems if it is not planned and controlled in relation to the traffic-carrying capacity of the interchange.

In the completed studies, attention was focused on the complexity of the problem related to the development of interchanges; land use controls now in force were analyzed; and suggestions as to the type of controls that might be effective were proposed. The author discusses these study findings and suggests that controls combining eminent domain techniques and police power might be the most appropriate ones for many areas. Suggestions also are made as to the guidance that State highway agencies could provide local communities confronted with interchange problems.

Areas requiring more research before definitive solutions can be formed are discussed; some are: Traffic generation characteristics of different land uses, service facility land requirements, compatibility of different land uses, effectiveness of different controls, and means for using available information to protect the capacity of the interchange.

Introduction

LAND USE problems are directly related to the amount and rate of land development. At present in the United States, the fast pace of land development is apparent and land use problems are becoming more numerous and serious. These land use problems are the result primarily of an increasing population, a rising standard of living, and an increasingly complex society (*1*).[2]

An opportunity and a problem

Land use problems in areas near the interchanges of controlled-access highways are especially numerous and intense. Interchange areas are not ordinary places. Because of the special advantages of accessibility, more economic activities are attracted to them than to areas not served by an interchange. This economic attraction constitutes both an opportunity and a problem, or at least a potential problem. There is almost universal agreement that the opportunity for economic development that interchanges offer should be used. Interchanges can serve as an instrument to open up new areas for sound economic growth, to revive the economic vigor of places that need economic revival, and perhaps to be the nucleus for a new kind of community. But this development needs to be planned so that it will continue to be economically sound over a period of time and so that the purpose of the interchange can be met.

Changes in land use occurring after a highway is built may impair the usefulness of the highway. Ribbon developments are an example. The goal of interchange area planners is to take advantage of the opportunity for economic development and at the same time guide or even restrain development that may be incompatible with the interchange or with other development in the interchange area.

Guidance for interchange development

Considerable attention has been given to land development problems and opportunities in interchange areas. Several planning groups and at least 17 State highway departments, in cooperation with the Bureau of Public Roads, have either conducted or sponsored research to find ways for maximum opportunities for economic development and to avoid land use problems in interchange areas. (A brief description of interchange studies sponsored by State highway departments and the Bureau of Public Roads is presented at the end of this article.) Analyses of all aspects of the interchange problem have been made in some of these studies and in others only certain aspects have been considered, such as the amount of space needed for highway-oriented businesses in interchange areas. Some of the more promising findings and recommendations from completed studies are described in this article, and a few suggestions are made for dealing with interchange matters. Also, attention is called to aspects of the interchange land use problem that appear to need additional research effort.

Summary

Location of a highway interchange near a community may or may not be beneficial and could be the source of a potential problem. Benefits from economic activity can be realized only when development of land use near an interchange is orderly; this requires planning. Information gathered in interchange studies made to date have been useful for—

(1) Focusing attention on land development in interchange areas; (2) providing a priority system that can be used to determine interchange areas where the need for establishment of some control on land use is critical; (3) showing the complexity of the problems in the interchange area, particularly those related to traffic generated some distance from the interchange; (4) analyzing land use controls—on this basis suggestions have been made on the possible use of new methods such as issuing licenses on a need-to-be-there basis or establishing criteria for determining traffic generation for different land uses; (5) evaluating the effectiveness of different land use controls in interchange areas—eminent domain was more effective than police power; and (6) providing guidelines that can be useful for determining space availability and space needs for highway services and other uses.

One of the shortcomings of some completed interchange studies has been the failure to consider what can be done in the absence of land use planning in the interchange area. For areas unprotected by local land use planning, it seems reasonable to freeze any

[1] Presented at the 44th annual meeting, Highway Research Board, Washington, D.C., January 1965.

[2] References indicated by italic numbers in parentheses are listed on p. 164.

future development near an interchange when the expected volume of traffic would exceed the capacity of the interchange ramps. Such a provision could be enforced in different ways; for example, by not permitting construction of additional driveways onto the feeder road near the interchange.

More information is urgently needed, particularly on other facets of the problem. Information is needed on the volumes and types of traffic generated by different land uses near interchanges, as well as the amount of interchange space needed for service stations, motels, and other uses. The compatibility or incompatibility of different land uses should be determined. Confirmation is also required as to whether it is good practice, as supposed, to have similar uses such as highway service facilities located in one quadrant of an interchange and different uses such as industrial facilities located in another quadrant. Information is also required as to whether well-planned, pleasant-appearing interchanges do in fact have a better economic future than poorly planned ones, and as to whether more attention should be given to planning for such highway needs as rest areas and placement of signs indicating the location of highway services.

Studies in depth, as well as surveys of broad coverage, are needed to gain the information required for adequate planning of land use controls at interchange areas. Surveys of a fairly large number of interchanges should be helpful in the selection of interchanges for analysis in depth, and information could also be obtained from the surveys on such aspects of the problem as number of driveway openings near interchange ramps and nearby highway service facilities. To the extent possible, research findings should be quantified; that is, given in percentage changes of land development, lineal feet or square feet needed for highway services, cost of controls through eminent domain procedures, and/or percentage of zoning appeals granted.

Interchange Development Problems

The nature and seriousness of interchange area development problems have been highlighted effectively during the past 4 years or so by individuals and organizations. One writer has described the freeway program as an ". . . unqualified success with the exception of the interchange areas . . ." (*2*). Another has likened freeways to people, suggesting that they both pass through youth, maturity, and finally old age. But interchanges reach the age of senility, he claims, more from the increase in traffic volumes than from the passage of time (*3*).

The economic problem that unsightly interchanges may pose for local areas and the opportunity interchanges afford for economic betterment often have been emphasized. A half-dozen States have issued booklets calling attention to the interchange as a valuable economic asset. For maximum benefit from an interchange, according to one account, planning must keep pace with economic development. If planning lags, the resultant development may be haphazard and poorly conceived. But no need exists to ". . . overplan for development, beyond any possible potential . . ." (*4*). Interchanges are the *new four corners* (*5*) or *your new front door* (*6*) and you cannot afford anymore to show your cluttered backyards, your garbage patches to America. So reads the literature intended to focus enough attention on the interchange area development problem to encourage constructive action.

One of the most successful efforts to highlight the interchange problem was the 1961 Highway Research Board Symposium on Land Use and Development at Highway Interchanges, reported in HRB Bulletin 288. The reports presented have caused much attention to be given to the highway interchange land use problem, and they also have provided an analysis of several aspects of the problem that still seems appropriate, such as the description and evaluation of controls, the description of the land to be demanded and supplied at interchanges, and the need for user services.

A complex problem

The true causes of the interchange problem are complex. Interchanges that have little economic development nearby may be congested with traffic; on other interchanges that are surrounded by development traffic may be moving satisfactorily. One of the most baffling aspects of this problem is the varying amounts of traffic generated some distances away that rely on the interchange for access. If it were not for the remotely generated traffic, the interchange problem would be almost as simple as permitting only those land uses that would not generate more traffic in the area than could be served by the interchange. Actually, even without the traffic generated from remote locations, two phases of the problem are formidable. Accurate information is needed on: (1) Normal traffic-generating characteristics and (2) traffic generated near the interchange but not using it.

A fairly common hypothesis made for most interchange studies is that, without some form of land use planning publicly enforced, interchange areas may be developed in an undesirable way—economically, esthetically, and in traffic generation. This hypothesis appears to be generally valid, but the fact that often it does not apply illustrates the complexity of the interchange area development problem. In a study in Texas, Professor Adkins failed to find any serious interchange problem in areas having no land use controls; these interchanges were subject only to market restraints. The results of the Texas study have caused some speculation concerning the possibility that the free market may work satisfactorily, and that land uses for which the owners can afford to be near interchanges will be suitable. In evaluating the absence of congestion reported in the Texas study, the effect of frontage or service roads must be considered. For example, ". . . in Houston, where there is no zoning ordinance and the Gulf Freeway is solidly lined with commercial and industrial uses, congestion at intersections was noticeably absent, even under the most heavy conditions of use. This might be partially attributed to the fact that this freeway, like many of those in Texas, has continuous collector-distributor roadways on either side of it which take a portion of the total transportation corridor demand." (*7*).

The complexity of the interchange problem is also suggested by the needs of different highway users. For example, one of the most commonly suggested ways of easing traffic congestion problems at interchanges is to limit access for some distance along the feeder road. But for highway users desiring to leave the highway only far enough to obtain food and fuel, such access control simply lengthens the distance the user has to go for this service (*8*).

To discern the relationship between elements of the complex land use interchange problem, several researchers have resorted to simulation models. Simulation models have been used in research completed or underway at the University of Washington, Pennsylvania State University, University of West Virginia, and the University of Virginia. Variables analyzed, in an attempt to predict development at interchange areas, have included population of the nearest urban place, population of the interchange area, traffic on the feeder road, age of the interchange, freeway capacity, land area available for development, and characteristics of the adjoining land. Some models have been used only to predict limited information, such as the need for service stations or motels. An adequate testing of these models has not been possible because of the lack of detailed information, especially data on traffic, land use, and population for the appropriate areas.

Development Controls Available

Problems related to attempts to control economic development by public decree were obvious as long ago as 1763, when King George III attempted to arrest the westward movement in America by forbidding settlements beyond the sources of rivers flowing into the Atlantic (*1*, Delafons p. 16). But difficulties of enforcement have not prevented land use controls from being used, and the adoption of land use planning measures has increased significantly in recent years. At present, more than 90 percent of the 1,355 cities that have a population of more than 10,000 people have an official planning agency (*9*).

Controls available in interchange areas have been surveyed often. Results of a 1960 survey by the Pennsylvania Department of Highways showed that, in the 36 States from which responses were received, little control of land use was provided explicitly for interchanges beyond extending access control some distance—from 100 to 1,000 feet—along the feeder road. In a 1961 survey on the land use protection provided at interchanges, only 17 States reported using some access control

on feeder roads (*10*). Data in this survey also showed that 22 States had no interchange land use controls planned or in effect. The nonurban nature of the State sometimes was cited as a reason for not providing guides or controls on interchange area development.

Differences in land use controls in urban and rural areas also were reported in a recent AASHO–NACO survey [3] of counties. Twenty-five pertinent questions were asked on such land use control techniques as zoning, subdivision regulations, driveway controls, setbacks, access regulations near interchanges, building codes, and mapped highway ordinances. Affirmative responses from urban counties were about 43 percent compared with about 22 percent for the rural counties. The 1,200 counties through which the Interstate System passes have more selected land use control measures than the other 1,800 counties; affirmative responses for the counties through which the Interstate System passes were 27 percent compared with 20 percent for other counties. Differences in land use controls by counties are shown in table 1.

Some interesting differences by States and regions were shown by the survey data. For example, the Western States of California, Washington, Nevada, and Oregon rank first, second, fourth, and fifth, respectively, in land use controls. As shown in figure 1 and table 2, Florida and Maryland also rank high, third and sixth, in having land use control measures.

Table 1.—States having land use controls, by percentage of counties responding affirmatively to 25 questions [1]

State	Responses from—			
	All counties in State	Interstate counties [2]		
		All	Urban [3]	Rural
	Percent	*Percent*	*Percent*	*Percent*
Alabama	13	14	26	11
Arizona	29	32	72	23
California	58	64	81	58
Colorado	28	36	55	28
Florida	35	38	52	28
Georgia	18	26	66	18
Idaho	14	22	41	16
Illinois	22	29	51	21
Indiana	25	35	45	32
Iowa	19	27	37	24
Kansas	16	30	55	23
Kentucky	14	19	42	16
Maryland	45	54	60	50
Michigan	25	28	30	25
Minnesota	22	26	34	24
Mississippi	11	16	32	14
Missouri	11	17	40	9
Nebraska	5	18	44	8
Nevada	35	40	54	35
New Jersey	31	31	31	29
New York	19	24	28	21
North Carolina	10	9	23	4
Ohio	31	40	59	34
Oregon	35	39	63	31
Pennsylvania	18	19	31	15
South Carolina	5	8	37	2
South Dakota	10	9	19	6
Tennessee	13	20	61	15
Texas	10	13	17	10
Utah	30	31	58	22
Virginia	28	34	64	30
Washington	48	51	64	45
Wisconsin	22	33	40	28
Wyoming	7	7	9	5
AVERAGE	22.4	27.6	44.7	22.3
MEDIAN	20	27	43	22

[1] Questions involved such techniques as zoning, building codes, long-range plans, driveway controls, subdivision regulations, setback ordinances, and mapped highway ordinances.
[2] Interstate counties are those crossed by or adjacent to an Interstate highway.
[3] Urban counties are those that: (1) are part of a Standard Metropolitan Statistical Area, or (2) had a 1960 population of 100,000 or more and had an increase in population of 50 percent or more from 1950 to 1960, or (3) had a population increase of 100 percent or more from 1950 to 1960, or (4) in 1960 were 70 percent or more urbanized (more than is typical for the United States).

Application of Controls

Everyone can agree that interchange areas should be neat and orderly. Nearly everyone would also agree that some type of public planning or land use control ordinarily is needed to assure that the development attracted to interchange areas proceeds in an orderly way. But there is no general agreement as to whether State or local governments should be responsible for guiding development in interchange areas; neither is there agreement on the general type of land use controls most appropriate for interchange areas, such as the use of police power or eminent domain.

Local or State controls

Land use controls such as zoning, subdivision regulations, and building codes generally have been administered by municipal or county governments. But land use controls available for use by local governments do not automatically ensure orderly development in interchange areas. Local land use protection devices may not be responsive to need for providing protection near highways, even though more than purely local interests are involved when an Interstate highway is built.

Apparently because of the problems arising when different agencies are responsible for building highways and for guiding development near highways, considerable thought has been given to providing land use control beyond or in the absence of control established by municipalities or counties. Many plans have been proposed or put into practice; they range from the existing situation in which land use controls generally are provided locally to plans for State administered zoning—sometimes accompanied by other police powers. Attention has also been given to possible establishment of interchange districts to guide development and highway strip zoning. In a few States, such measures have been enacted or are being seriously considered.

A standard feature of any plan to provide land use controls, either through interchange districts, strip zoning, or other State action, is use of local initiative to the fullest extent possible. This can be done in several ways: By excluding local areas or interchange districts from Statewide zoning if local planning of development is satisfactory; by changing local control only to the extent of referring rezoning questions to a State agency; by giving local governments a specified period of time to zone or provide other land use controls in interchange areas; by leaving zoning matters generally to localities, except where local practice and State interests are clearly incompatible.

To combine State leadership with local initiative, local interests must be considered as soon as possible. Highway plans are more likely to be respected by local groups if these plans have been made a part of the local land use plans. This has been proved in different locations. In Duluth, Minn., for example, planning of several interchanges on Interstate 35 has been integrated with local planning, and local streets have become frontage roads along part of the Interstate System. Coordinated planning of this type, in which city planners share in the decisions on highway location and alinement, is expected to continue to receive local support. This type of planning will contribute to orderly development in the interchange area. Other areas where effective coordination of interchange and local planning have been reported or planned include the State of Illinois and Tulsa, Okla. Result of

[3] For this survey by the American Association of State Highway Officials and the National Association of County Officials, much of the tabulation and analysis was done by Phillip Patterson, a student employee of the Bureau of Public Roads during a part of 1963 and 1964.

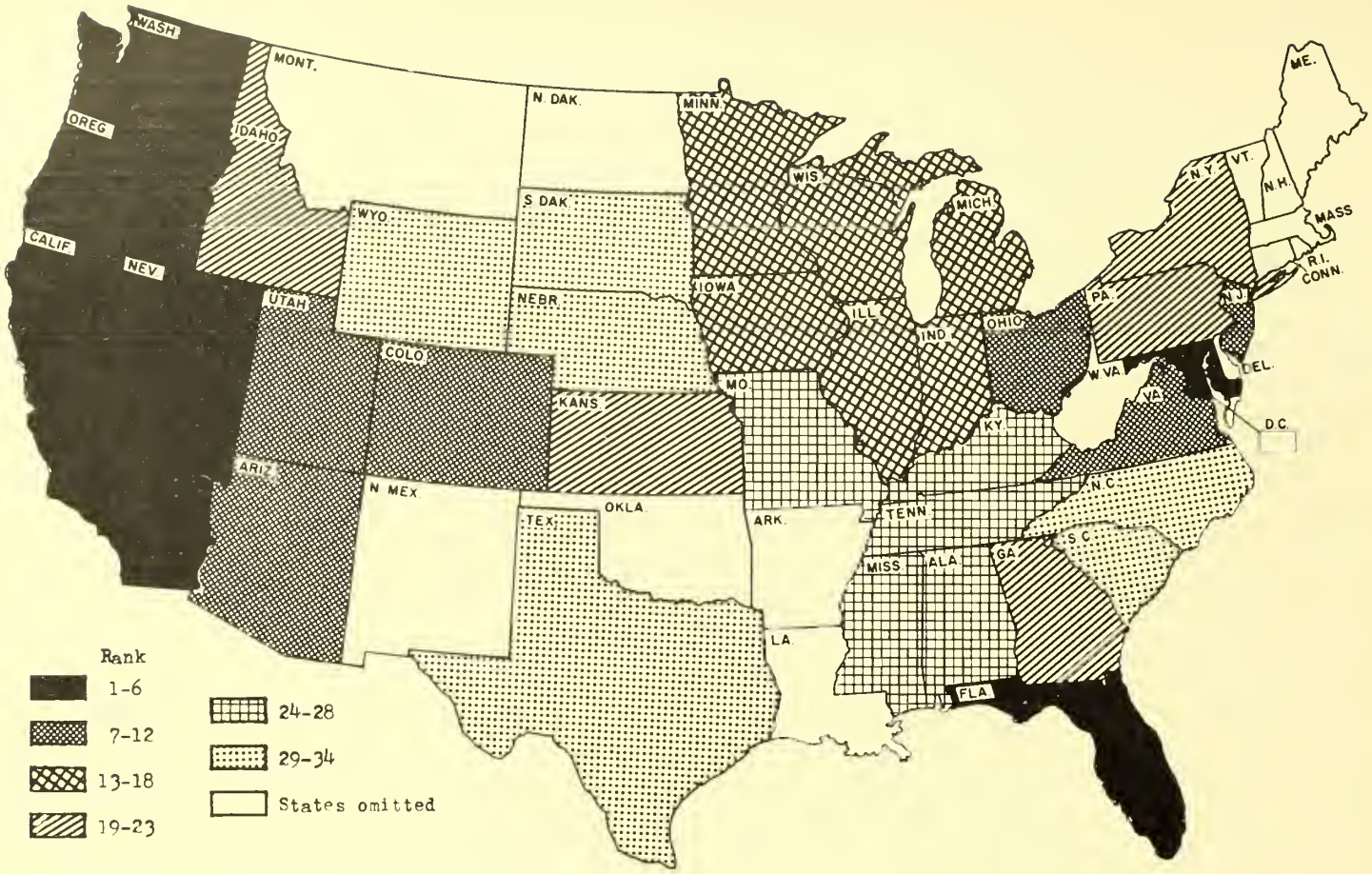


Figure 1.—Land use controls available to counties—rank of 31 States by responses to questionnaire.

the coordination in Tulsa has been the fairly general acceptance of having driveway plans approved and administered jointly by the local planning commission and the Oklahoma Department of Highways (*11*). In Illinois, highway and local development plans have been proposed (by Barton-Aschman Associates) for selected interchange areas; these plans are intended to provide maximum opportunities for local development and to facilitate service for highway travelers.

Staff requirements

One of the biggest problems related to any land use control plan involving States is the acquisition of a staff large enough and experienced enough to make the plan successful. Therefore, the best role for States may be the responsibility for those aspects of interchange control in which they have experience, such as the extension of access control along the feeder road, and for providing information and leadership to enable local, land use control authorities to do their jobs more effectively. Relevant information would include early word about such things as route locations, traffic forecasts, design capacities, trip generating characteristics of different land uses. Elements of leadership that can be supplied include a clear understanding of the intended purposes of a highway in a particular area, such as for tourist travel, commuting, or bypassing urban traffic congestion, and ways that localities can obtain maximum benefits from the opportunities afforded by nearby highway facilities. Sometimes, for example in Alabama and Pennsylvania, this leadership has been provided in part by State university spokesmen, especially from universities involved in analyzing the economic opportunities afforded by nearby highways.

Eminent Domain and Police Power

The type of controls that might be most appropriate for interchange areas also has received much attention. Of all the legal techniques available, only the power of eminent domain and the police power have had much use along highways. The police power has, of course, been used more than the power of eminent domain in the control of development in interchange areas.

The police power, involving a reasonable regulation of property use in which no payment goes to the owner, is exercised through such measures as zoning, subdivision regulations, mapped streets, setbacks, and driveway controls. Zoning is the most common and best-known control used. Its problems and shortcomings are well known. In brief, zoning controls generally cannot be maintained for long against economic pressures. Zoning has been likened to weeds in a garden requiring ". . . constant attention which is difficult to sustain over a long period of time. . . . Zoning is vulnerable to review, change, and pressure from special interests." (*12*).

Zoning problems

These zoning problems have been well documented. For example, in the AASHO–NACO survey previously referred to, Montgomery, Prince Georges, and Howard Counties—three fast-growing counties in Maryland that are crossed by an Interstate highway—appear to have nearly all of the land use controls listed on the questionnaire. But the existence of these controls has not prevented areas in these counties from having problems in land development. That these zoning problems may be fairly common is suggested by information from a University of Washington study, as well as other studies. Information concerning the disposition of rezoning applications from some 41 municipalities during the survey period shows that an applicant for a rezoning had a 61- to 80-percent chance of getting what he wanted in the first application. Information was reported in the study that ". . . the situation is similar for rezoning activity adjacent to or near interchanges in six cities and counties studied in detail through field work." This problem of rezoning is, of course, only a part of the overall problem. Fully as important is the problem of inadequate enforcement of zoning and other land use control measures. As stated in the Washington

University study report, ". . . typically, building departments and planning staffs do not make systematic checks of land use against the zoning map. Public prosecutors act only when requested to do so either by the administrative unit concerned or by a complaining citizen." (*12*, pp. 10–23.)

New Orleans and Philadelphia had a similar pattern of zoning appeals and exceptions. Of 963 cases studied in a recent year in New Orleans, 90 percent were granted. Of 256 cases heard in Philadelphia, only 47 were denied. Interestingly, for the 30 persons appearing in person, each appeal required an average of 18 minutes for hearings compared with 3⅓ minutes on appeals when no person appeared (*1*, pp. 88–90, Delafons).

The advantages of having zoning controls are also obvious. Zoning can slow the pace of development until more intensive measures can be provided, and it can affect the density of development. Zoning can also be especially effective in helping to stabilize land use after redevelopment of an area.

Subdivision regulations, another police power, are also being used to a considerable extent, though not so widely as zoning. Subdivision regulations are used primarily because they affect developers at a time when they are still able to make changes in the subdivision and then pass any increased cost along to the buyers.

The power of eminent domain, involving the payment of public money for certain property rights, can also be effective for controlling land usage when it involves such techniques as development rights, easements, purchase and leaseback, and condemnation of excess land. The primary advantage of eminent domain is simple enforcement. Because an agreement by which landowners transfer redevelopment or other rights has the power of a contract, violations of these rights are not likely to occur.

Table 2.—Rank of States according to land use controls in effect, by counties [1]

State	Responses from—			
	All counties in State	Interstate counties [2]		
		All	Urban	Rural
	Rank	*Rank*	*Rank*	*Rank*
California	1	1	1	1
Washington	2	3	4	3
Maryland	3	2	8	2
Nevada	4	5	13	4
Oregon	5	6	6	7
Florida	6	7	14	11
Ohio	7	4	9	5
New Jersey	8	13	26	9
Utah	9	14	10	18
Arizona	10	12	2	16
Colorado	11	8	11	10
Virginia	12	10	5	8
Indiana	13	9	16	6
Michigan	14	18	28	13
Wisconsin	15	11	20	12
Illinois	16	17	15	19
Minnesota	17	20	24	15
Iowa	18	16	22	14
New York	19	21	29	20
Georgia	20	19	3	21
Pennsylvania	21	25	27	25
Kansas	22	15	12	17
Idaho	23	22	19	23
Kentucky	24	24	18	22
Tennessee	25	23	7	24
Alabama	26	29	30	27
Missouri	27	27	21	29
Mississippi	28	28	25	26
Texas	29	30	33	28
North Carolina	30	31	31	33
South Dakota	31	32	32	31
Wyoming	32	34	34	32
Nebraska	33	26	17	30
South Carolina	34	33	23	34

[1] Tabulated from the number of affirmative responses made to 25 questions on land use controls, involving such techniques as zoning, building codes, long-range plans, driveway controls, subdivision regulations, setback ordinances, and mapped highway ordinances.

[2] Interstate counties are those crossed by or adjacent to an Interstate highway.

Disadvantage of eminent domain

The main disadvantage of controlling land use by eminent domain is the high cost. For example, costs for redevelopment have ranged from 13- to 84-percent of the market value in fee simple along parts of the Mississippi River Parkway in Wisconsin and along some parts of the Natchez-Trace Parkway in the South (*12*, p. 37). And, an easement forbidding high-rise apartments on a 47-acre site along the Potomac River near Washington, D.C., was valued at $750,000 by a jury even though the cost of the property in fee simple had been only $650,000 a short time previously. When the cost of purchasing certain rights, in order to control the use of the land, approaches the cost of purchasing the property in fee simple, generally it is preferable for the public agency to purchase the property outright; later, if all or part of the property is sold, development controls such as restrictive covenants can be imposed.

Best controls

A combination of police power and eminent domain techniques appears to be the best answer for controlling economic development near interchanges. Professor Horwood suggests a combination of zoning, land redesign, and the acquisition of access rights. Acquisition of access rights ordinarily is less expensive than most other eminent domain techniques of land use control, and this control can be effective if used carefully. For example, access rights can be purchased to control industrial or other uses considered potentially hazardous to interchange traffic capacity, and the landowner will be free to develop the land for some other use. An example of an apparently successful combination of the police power and eminent domain has been reported from Tennessee. There, if a building permit is requested that would interfere with future highway building, the land involved is acquired by the highway agency (*13*). Such a fusing of police power controls and eminent domain seems to provide optimum protection for both the public interest and the private property owners' rights.

Certain land use controls that are being adapted from other situations also may be helpful in the interchange problem. For example, a concept has been formed from studies in Minnesota and Illinois that only those uses requiring highway location should be permitted in such locations. This concept has a precedent in the formation of Waterfront Districts, by which a harbor or other such facility is recognized as being a natural resource in which the community has a vested interest. In the same way, an Interstate highway could be recognized as a transportation resource in which the entire community has an interest. Another possible approach, which is not new, is to use development performance standards to limit the amount of traffic by controlling large traffic generators. These performance standards could be developed along the lines of the standards now employed to determine when establishments exceed permitted generation levels of smoke, sewage, odors, or noise.

Priorities for Interchanges

Providing land use planning for all interchange areas seems to be an almost overwhelming task. However, the task becomes manageable when it is taken up one part at a time. If the parts of the interchange problem are to be taken in logical order, suburban interchanges near the fringes of urban centers deserve attention first. Near these interchanges, development of land is more likely to be uncoordinated than in urban or rural areas. In rural areas, land development problems can be expected to be fairly mild. But some pressure for economic development may be exerted for land near urban interchanges. However, near urban interchange areas, general land use controls are probably available

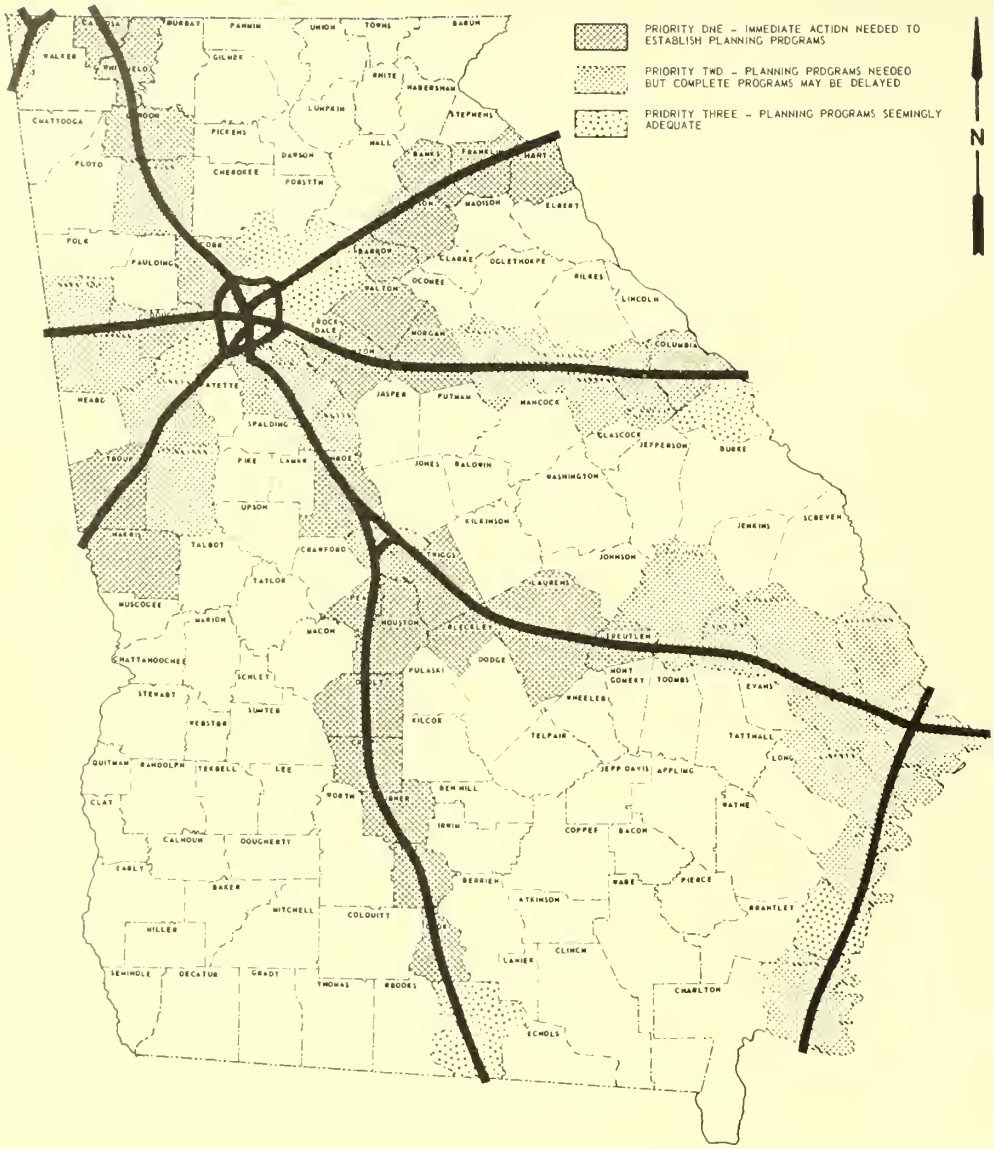

PRIORITY ONE – IMMEDIATE ACTION NEEDED TO ESTABLISH PLANNING PROGRAMS
PRIORITY TWO – PLANNING PROGRAMS NEEDED BUT COMPLETE PROGRAMS MAY BE DELAYED
PRIORITY THREE – PLANNING PROGRAMS SEEMINGLY ADEQUATE
N

Figure 2.—Priorities for county planning in Georgia for land use planning controls along the Interstate System.

and the use permitted of the surrounding land probably has been established.

The usefulness of a system of priorities that focuses attention on the interchanges and on the land use problems that need solution first have been demonstrated by the procedure followed in the Georgia study. Interchanges were analyzed according to their location (urban, rural, suburban) and the type of crossroad (primary, secondary, etc.); from this information a priority system was developed to indicate how critical the need was for county planning. Counties in priority one are those in which parts of the Interstate System have been completed or are due for completion by 1965 and in which unincorporated areas are not subject to planning commissions. Priority two generally includes counties in which none of the Interstate System will be completed until after 1965. Priority three includes counties that have programs deemed adequate for controlling land use. Fast action on the interchange land use problem is needed: 50 percent of the 58 Georgia counties through which the Interstate System passes have been placed in priority one. That is, the Interstate System passes through these counties but they do not have adequate land use planning protection. Only 10 percent of the 58 counties were judged to have adequate land use control programs (*14*). The priorities assigned to Georgia counties crossed by the Interstate System and Georgia counties that have planning commissions are shown in figures 2 and 3.

Priorities of a somewhat similar type have been developed in studies in Pennsylvania, Michigan, and Minnesota, and by the committee of AASHO–NACO, referred to previously. In Pennsylvania, interchanges have been listed according to whether they will be located in cities, boroughs, first class townships, or second class townships. In addition, highway locations and county planning commissions have been related to one another on maps, as illustrated in figure 4.

The interchange priorities system for Michigan involves redesign and reconstruction of interchanges rather than land planning in the surrounding area. Some of the conditions justifying redesign could have been prevented or alleviated sooner by land use planning; for example, a sudden jump in traffic volume caused by the change in nearby land use could have been prevented or controlled by land use planning (*3*, Neve, p. 3).

In the Duluth, Minn., study already referred to, a priority system has also been established for land use planning. This priority system consists of: "Immediate Actions Prior to Interstate Construction; Actions with Interstate Construction; and Future Actions." (*15*). And the AASHO–NACO survey of interchanges has provided general information showing the land use planning techniques being used in fast-growing and/or urban areas having Interstate routes. The interchange land use problems in these counties might be regarded as of high priority because interchange problems in these counties are expected to be more critical than they are in rural areas.

Space Needs at Interchanges

The crux of the interchange problem seems to be how much of the land made available for development at interchanges will be used and the type of use to which it will be put. Professor Garrison has estimated that by 1980 intensive land development will occur near the interchanges of large metropolitan areas but that in small urban centers the available land near interchanges will exceed the demand (*16*). Space needed for residential development is expected to far exceed that for the other specified uses—industry and shopping centers. Although the space needed for shopping centers is by far the smallest of the three uses, it needs to be carefully coordinated with interchange plans because of the traffic generating characteristics (large volumes) of shopping centers.

Highway services

Space needs for highway services in interchange areas have received special attention. For example, in studies in Minnesota the amount of space used for highway services has been related to the traffic volumes along the highway. By using this information and traffic projections for 1975, the amount of space needed for highway service near certain interchanges has been estimated. For example, an expected 45,850 average daily traffic in 1975 will require about 4,400 lineal feet for highway service at one interchange in Duluth.

Pennsylvania researchers have gathered information from interchange sites concerning the relative frequency of service stations, restaurants, and motels. This relation was 21 : 14 : 10—that is, 21 service stations to 14 restaurants to 10 motels. And in a study in Wisconsin, auto service and food service were reported to be 66 percent of all roadside

establishments (*10*, p. 43). Obviously existing space allocations near interchanges can be of only limited usefulness in estimating space needs for highway services because of the apparent tendency of some oil companies to obtain more space for service stations than will be needed in the immediate future.

In addition to the amount of space allocated to highway services, interchange land use planners need to know the preferred location of highway services in relation to the interchange. Several studies have been completed that indicate the most desirable location for highway services is the first quadrant of an interchange on the right side of an urban area exit. Apparently this location is favored because of the ease with which service facilities can be reached without the need to make left turns, even on a diamond interchange. The first quadrant on the right side of an approach to an urban center also is desirable (*4*, p. 13).

Frontage roads, in some places, have been used in an efficient way to provide space for highway services—such usage has been reported in the Duluth area and in many areas in Texas. In some places the frontage roads also serve as traffic reservoir areas, which helps to alleviate traffic backup on exit ramps (*17*). But a continuing problem exists in relation to frontage roads: They may magnify roadside control problems in nonurban areas where strict land use controls are not in effect.

An Answer for All Interchanges

The bulk of the 14,000 Interstate interchanges will be located in rural areas. In many of these rural areas, little land use planning exists beyond crop rotation, and no agency or individual is available to administer land use planning legislation even if it were to be enacted.

Some thought has been given as to what can be done to provide effective land use control in the absence of land use planning by local authorities. In some States, such as California and Florida, legislation has been enacted to establish interchange districts on a limited basis. In other States, the State highway agency is relied on to help administer or police subdivision regulations, roadside zoning, driveway controls, or some similar land use control measures. Another land use control measure considered for application to rural interchange areas would have the State freeze existing land use near interchanges until acceptable land use controls could be established, either by the local government or the State. The effect of such a freezing or fixing of existing land use would probably be very similar to the effect of agricultural zoning. Control of land use either by creation of interchange districts or freezing probably would curtail and delay land development activity near rural interchanges.

An alternate plan would permit economic development in an interchange area until traffic on the feeder road and/or at the interchange approached the design volume. When this occurred, it seems reasonable to expect that highway builders would take action to prevent further deterioration at the interchange. The action to take is a problem. In the past, development of land use near interchanges has not been sufficiently controlled and the solution to the resultant traffic problems in some places has required that the interchange be redesigned. A better approach would seem to be the imposition of measures to control development beyond the expected accommodation of the highway facility, perhaps by permitting no additional driveway openings onto feeder roads within a certain distance of the interchange. Controlling the number of driveways on feeder roads is obviously a modest approach to the land use problem near interchanges. Local authorities could misplan or ignore development near interchanges until the land uses nearby generated enough traffic to overwhelm the interchange. This approach, therefore, is not worthwhile as a substitute for more adequate controls already in existence or planned. This approach has been suggested for use only where control is lacking—a situation existing at present for many rural interchanges.

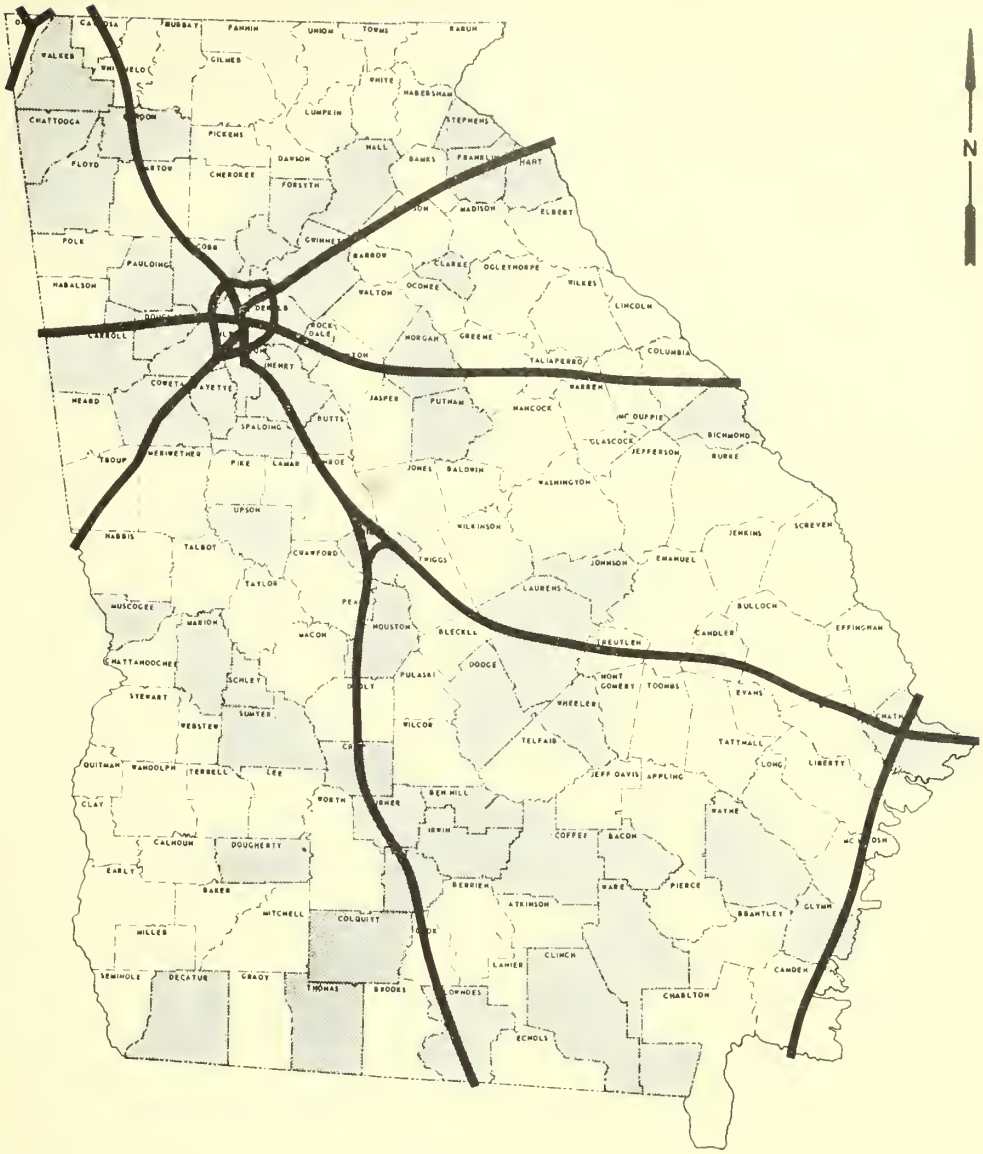


Figure 3.—Georgia counties having official county or joint city-county planning commissions.

Restraining development only at those rural interchanges where traffic congestion is imminent would be advantageous because land planning effort could be concentrated on interchanges where attention is needed. No appreciable development may occur near many rural interchanges, and it is unrealistic to expect that a land use plan will be developed for each interchange. Although a land use plan in sparsely populated rural areas might be very simple, any effort to be expended for controlling roadside development probably needs to be channeled to locations where problems of interchange land use are most urgent.

A serious disadvantage in a hands-off type of approach to land use controls at interchanges is the possible development of junkyards or other eyesores near rural interchanges. Hence, more effective land use planning should be encouraged wherever its use is

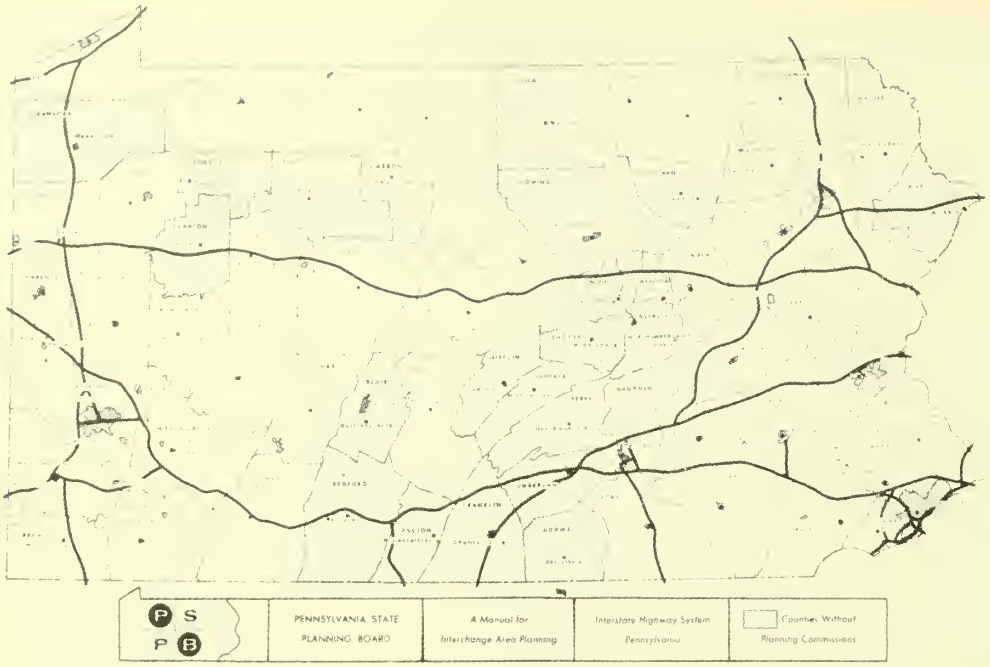

PENNSYLVANIA STATE PLANNING BOARD
A Manual for Interchange Area Planning
Interstate Highway System Pennsylvania
Counties Without Planning Commissions

Figure 1.—Interstate highways in Pennsylvania and counties not having planning commissions.

realistic. But, if past experience is any guide, land use planning can be expected to continue to lag behind highway construction.

For urban and suburban areas, nearly any combination of control devices described heretofore can provide a satisfactory solution to the problems of land use near interchanges, if they can be enforced. To guide the development near interchanges effectively in either urban, suburban, or rural areas, more specific information is needed. Local authorities have a right to expect highway builders to furnish much of this information, such as traffic generating characteristics of certain land uses, design capacities of highway facilities, and compatible and incompatible land uses in interchange areas. Several States, through their interchange pamphlets or manuals, movies, speakers, and other media, have provided communities with at least some of the vital information, such as driveway spacing and service roads, as shown in figure 5.

Existing land use controls apparently would be more effective if better cooperation could be achieved between interested agencies.

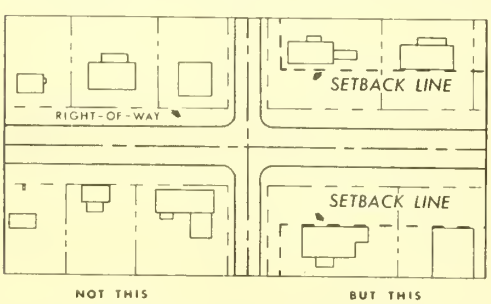


Figure 5.—Desirable type of setback (from A Manual for Interchange Area Planning, Pennsylvania State Planning Board).

For example, at a recent rezoning hearing on an application for a permit to construct a 16-acre amusement park and swimming pool on a busy approach road to a major highway facility, only local residents and the amusement park builders appeared. Perhaps highway builders could assume more responsibility for assuring that generators of large volumes of traffic do not encroach on highway facilities. Another possibility for cooperative action could be among the Federal Housing Authority, the Bureau of Public Roads, and the States.

Possible Solutions

The interchange research completed so far obviously has not provided all the answers needed to solve the problem of land use near interchanges. Some of the solutions recommended in the studies have been different. For example, in the report on the Georgia study land use planning was recommended primarily at the local level, but in most studies it is recommended that such an important responsibility be lodged with a State agency. One conclusion was made in all the reports: that more needs to be learned before the problem can be solved. The absence of criteria and guidelines for applying the different legal techniques available, in order to reach the overall goal of control of land usage near interchanges, is one of the obvious gaps.

General agreement seems to have been reached as to the ultimate goal: interchanges on which traffic can be expected to move in an orderly and efficient manner. A fairly satisfactory understanding also exists as to the different land use control measures presently available, such as zoning and redevelopment rights. But the principles of desirable land use arrangements near interchanges are not completely settled. It is almost as if the *way* to plan (the legal techniques) has been learned but *what* to plan (the desirable arrangement of land use) is still unknown. The urgent need now is specific information about land uses that are suitable for interchange locations and that also are good neighbors in an interchange setting, trip generating characteristics of land uses near interchanges, space needs for highway services, and ways of implementing uses that research has demonstrated are suitable.

Trip Generation Characteristics

Most studies of interchange development have involved some attempt to determine whether traffic using the interchange can be accommodated. Such an analysis requires fairly specific information about trip generating characteristics of land uses served by the interchange, the type of area served by the interchange, and traffic capacity of the interchange. In the past, problems have been encountered in getting specific information on any of these three items, including traffic assignment data (*7, 18*). Information now available is still far from satisfactory. For example, general agreement has been reached that the interchange area should be the area of origin and destination for trips using the interchange. Although this concept is pure, it is unwieldly in use, and some zone based on experience for typical interchanges probably will have to be used in defining the interchange area.

Information about trip generation characteristics of different land uses near interchanges is becoming available, at least on a fragmentary basis. Although little specific information is available concerning highway user services such as service stations, motels, and eating places, these uses rarely have a traffic generating pattern that conflicts with the urban work trip. In at least one study, the report showed that less interference to traffic was caused by fewer vehicles turning into service stations than those turning into restaurants, motels, residential, and commercial areas. Fifteen percent of the turns made for service stations caused interference compared with 24 percent of those made for other uses (*19*). Some of the reasons for this difference in interference are obvious, such as open spaces around service stations and a sufficient number of service stations so that left turns and waiting to enter can ordinarily be avoided.

Shopping centers

Some trip generation information pertinent to interchange planning is also available for shopping centers. Regional shopping centers that stay open in the evenings apparently have their peak traffic at about 8 p.m. (*20*) and neighborhood shopping centers seem to have their peak traffic loads earlier in the evening (*21*). Shopping centers need to be " . . . a substantial distance from ramp ends . . ." and they should be placed where left turns can be avoided on the trip home from work (*14*). A shopping center located

Figure 6.—Example of inadequate access control on a crossroad beyond the end of a ramp. Photo shows clustering of church, residential, and shopping center developments at Stevens Creek Road interchange near San Jose, Calif. Hazardous entrance to Valley Fair shopping center (arrow 1) where vehicles make right turns into parking lot in front of traffic coming from the freeway ramp. Hazards (arrow 2) would be substantially reduced if access had been prohibited for 500 feet beyond the end of the ramp; the distance to the entrance is only 240 feet.

too close to a ramp exit is shown in figure 6. Potential congestion is, of course, more dangerous on exit ramps than on entrance ramps.

Traffic generation

The urban transportation studies now underway are expected to be a useful source of information about traffic generation characteristics. For interchanges at different distances from large urban centers, it seems important to have trip generation information by density as well as by land use. Such information from urban studies made in Chicago, Ill.; Pittsburgh, Pa.; Detroit, Mich.. and Minneapolis and St. Paul, Minn., is summarized in figures 7 through 11. Density, or distance from downtown, as shown, seems to have had a noticeable effect on most of the uses classified. In addition to being a valuable source for trip generation data, the information being developed in urban transportation studies on comprehensive planning also should be helpful in the attempts being made to solve interchange area development problems generally, including those related to interchange areas outside urban areas. Also, experience being gained through the Urban Renewal Administration's Workable Programs for Community Improvement may prove to be appropriate and helpful in solving interchange area development problems (*22*).

For analyzing and anticipating land development problems at an interchange, information on traffic generation characteristics for nearby land uses obviously must be supplemented with information about the amount of traffic that actually uses the specific interchange and the percentage of crossroad traffic on the feeder road into the interchange. This percentage can vary; for example, up to 100 percent if the feeder road deadends at the interchange (*10*, p. 47).

To spread the traffic burden among nearby interchanges, a variety of land uses might be desirable. However, a variety of land uses at one interchange area might create another problem—make shoppers travel further. This problem also requires more research.

Compatible and Incompatible Uses

Many of the land use problems at interchanges, it is hoped, can be solved by excluding incompatible uses from the area. The general principle of compatible and incompatible uses has been applied previously, that is, prior to the time of the problem of land usage at interchanges. The application of this principle presents some problems and emphasizes the need for more research. At present, information available on compatible uses seems to be somewhat contradictory, no doubt because it is incomplete.

An axiom in city planning theory is that like things belong together. For example, food wholesalers located together permit careful buyers to keep their travel at a minimum while they visit several establishments. Such highway-oriented establishments as service stations, restaurants, and motels are considered to be compatible because they have common customers. Having such compatible uses near one another, preferably in the same quadrant of the interchange, seems to be reasonable as a goal and has been achieved in many locations.

The principle of grouping like or compatible uses together should probably be applied to interchange areas with some restraint, because it is obvious that ". . . a land use arrangement which has a variety of uses is superior to one which has segregation of uses; variety spreads the traffic burden while segregation concentrates it." (*23*). For example, a factory and a machine shop may be compatible but their proximity to one another and to an interchange may aggravate traffic problems during the morning and evening peak traffic periods. From research findings already available, some determinations have been made on ways to reconcile these two principles: that like uses are compatible, but that a variety of uses spreads the traffic burden. If different quadrants of an interchange are

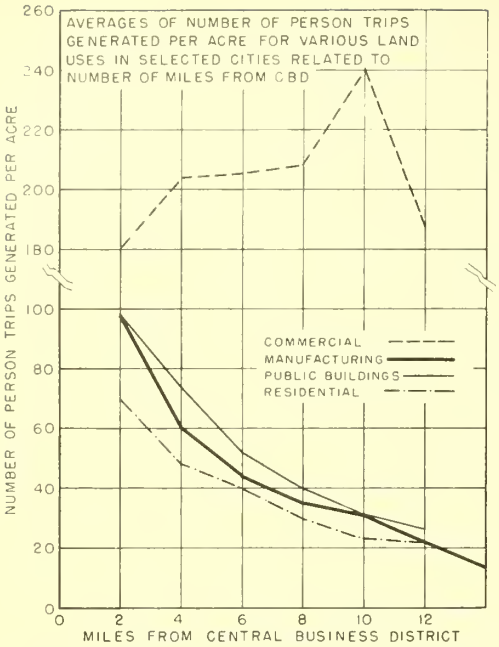

AVERAGES OF NUMBER OF PERSON TRIPS GENERATED PER ACRE FOR VARIOUS LAND USES IN SELECTED CITIES RELATED TO NUMBER OF MILES FROM CBD
NUMBER OF PERSON TRIPS GENERATED PER ACRE
COMMERCIAL
MANUFACTURING
PUBLIC BUILDINGS
RESIDENTIAL
MILES FROM CENTRAL BUSINESS DISTRICT

Figure 7.—Traffic generation by combined land uses.

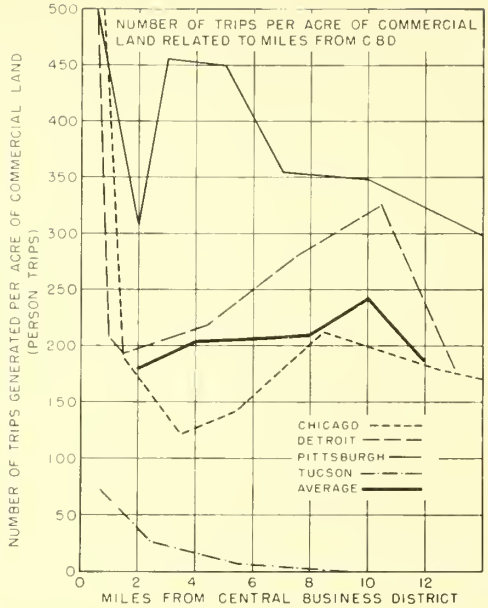

NUMBER OF TRIPS PER ACRE OF COMMERCIAL LAND RELATED TO MILES FROM CBD
NUMBER OF TRIPS GENERATED PER ACRE OF COMMERCIAL LAND (PERSON TRIPS)
CHICAGO
DETROIT
PITTSBURGH
TUCSON
AVERAGE
MILES FROM CENTRAL BUSINESS DISTRICT

Figure 9.—Traffic generation by commercial land uses.

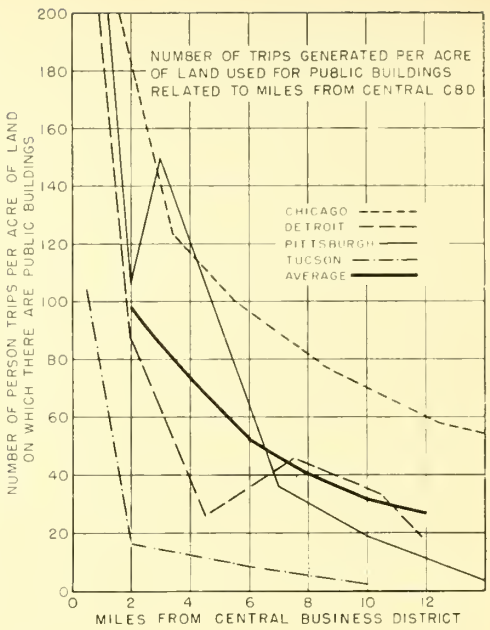

NUMBER OF TRIPS GENERATED PER ACRE OF LAND USED FOR PUBLIC BUILDINGS RELATED TO MILES FROM CENTRAL CBD
NUMBER OF PERSON TRIPS PER ACRE OF LAND ON WHICH THERE ARE PUBLIC BUILDINGS
CHICAGO
DETROIT
PITTSBURGH
TUCSON
AVERAGE
MILES FROM CENTRAL BUSINESS DISTRICT

Figure 11.—Traffic generation by public buildings land uses.

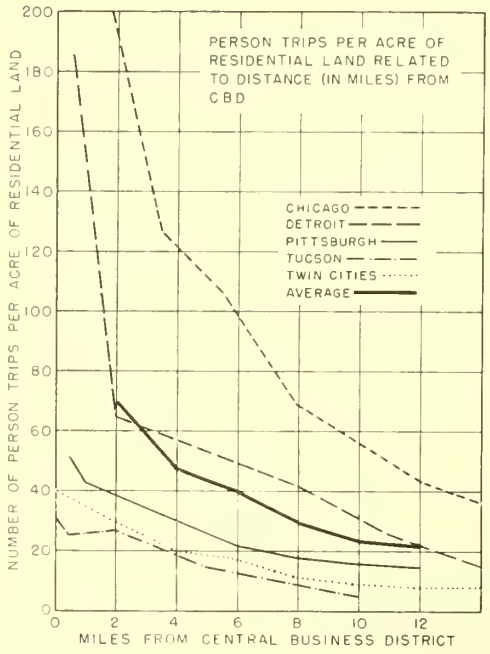
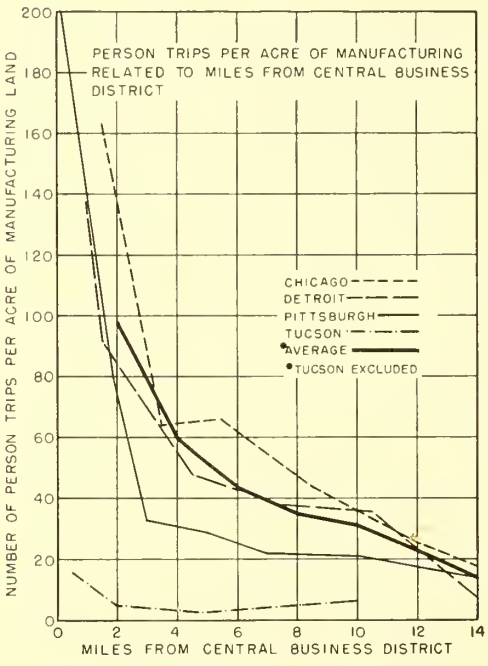


Figure 8.—Traffic generation by residential land uses.



SOURCE: AVERAGES DERIVED FROM DATA IN THE CHICAGO, DETROIT, PITTSBURGH, TUCSON AND TWIN CITIES TRANSPORTATION STUDIES.

Figure 10.—Traffic generation by manufacturing land uses.

put to different uses, both principles apparently can be served. Thus, perhaps such low traffic generators as motels can be located in one quadrant, and a factory, which generates heavy peak traffic loads, can be located in another. Such an arrangement would not be in accord with the concept presently receiving some attention, that land use for some interchanges should be limited to highway services and for others, to community services.

More research is needed to obtain additional information about the conditions under which certain uses are compatible or incompatible. The general belief that highway user service facilities are compatible may need to be revised. At one interchange, an overnight facility has been established for horses being transported by trailer or truck. Although such land use is apparently a highway-oriented business, it is probably incompatible with businesses such as motels and restaurants. Another development complicating the concept that highway-oriented businesses are compatible is the use being made of motels as community and convention centers, as well as tourist stops. The traffic generating characteristics of motels providing these services undoubtedly differ from those of motels catering only to highway travelers.

The concept of compatibility can also be applied to compatibility between the interchange and certain types of use. Some businesses established at interchange locations do not need to be there and, in fact, may be detrimental to the use of the interchange or the business may be affected adversely by such a location. Although auto junkyards are generators of low volumes of traffic, they obviously should not be permitted near interchanges. Also, locating schools and churches near interchanges may be less desirable than selecting locations along a controlled-access highway between interchanges, where access can be gained away from the highway or from a frontage road. Such a location would permit the institution to realize the benefit of being on display along a modern highway, of having an easily remembered address—near a certain interchange or exit number—and of being easily accessible to people walking in the neighborhood.

Space Needs Near Interchanges

Additional research is also needed to determine whether user services or other vital needs are being satisfactorily anticipated. There is general agreement that space for user services should have priority and that other uses, such as residential, should not preempt it. Because of the aggressiveness of oil companies in acquiring building sites, no danger of a shortage of space for service

Figure 12.—Cars parked near an interchange by members of carpools.

Figure 13.—Parking on feeder road by members of carpools.

stations is apparent. But space requirements for other highway services are not being cared for so well. For example, space for off-street parking at strategic locations near interchanges is needed for people assembling for carpools, a practice that is becoming fairly common in outer suburban areas near large cities. This land use need should be recognized and met. The current practice of parking haphazardly along the sides of feeder roads, as illustrated in figures 12 and 13, has had several bad effects. For example: Snow removal and maintenance have been impeded; landscaping has been damaged; capacity has been reduced on the crossroad; and the accident potential has been increased because the drivers' sight distances have been reduced. Little specific information is available on this problem. Information from one study indicates that space is needed for 5 to 10 cars near an interchange where such parking is done. Those parking near an interchange typically travel 30 miles to work, one-fourth of it before the interchange is reached. The average membership of carpools assembling at an interchange is 3.5 members (*24*). Encouragement of suburban parking near an interchange and the resultant use of carpools should alleviate downtown traffic problems.

Research needed

Additional research effort is needed to determine generally the future space requirements for such highway user services as motels and service stations. Information about apparently workable solutions to the problem of parking near interchanges should also be gathered and analyzed. For example, a few shopping centers now welcome all-day parkers. This solution, though not applicable to land uses near interchanges in outer suburban areas, seems promising as commuter parking normally ends between 5 and 6 p.m., before peak shopping center parking requirements occur between 6 and 8 p.m.

Techniques to Implement Interchange Planning

Local governments sometimes are criticized for their failure to provide adequately for orderly development in interchange areas. But local officials may be busy on other problems and cannot always act to establish controls on development suddenly caused by highway construction. Furthermore, the facts needed by a local government before it can deal effectively with some of the land use problems in interchange areas often must first be provided by a sophisticated regional planning operation, from urban transportation studies, or from a State highway agency.

Sometimes, however, available information establishes the fact that local land use control would be desirable. A few of the incentives used to encourage a local government to act have included: (1) State grants to localities to cover costs of administering land use controls; (2) State highway improvements that are conditioned on the existence of an acceptable local land use plan; and (3) tax relief for developers who use good design, when good design can be agreed upon (*25*).

As suggested previously, one of the best ways to encourage local land use planning is to provide the information and technical assistance necessary. Undoubtedly, provision of information as it becomes available to illustrate general truths about transportation and land use relationships would be worthwhile. Some examples of this type of information are: (1) highways sustain land values only so long as there is a reasonable balance between the capacity of the highway and the uses to which the land is being put or (2) good esthetics are good economics. In studies in Pennsylvania and Texas, the close relationship existing between good esthetics and good economics has been especially emphasized, as has the fact that unsightly interchange areas may not attract the same tourist trade as the well-planned land use near other interchanges. The close relationship between high development standards and sound economic development was demonstrated convincingly in a Dallas study that includes information about Richardson, Tex.; this relationship is apparent when the economic development of comparable towns is considered along with their lower development standards (*26*).

Quantification of Data

Some of the information from interchange area research has been presented in numerical form. Such quantification, though it may have limitations, permits comparison between study findings in different locations and ultimately should facilitate certain administrative decisions. The start that has been made, for example, in comparing redevelopment costs with the cost of acquisition in fee simple obviously could be useful in making a decision as to the approach to use in guiding development (*12*, p. 37).

In another analysis, the cost of not having some control device, such as roadside zoning, was reduced to a numerical value by applying the 5 cents of each highway dollar spent to remove structures from highway right-of-way to the $500,000 being spent to modernize the system over a 20-year period. The resultant $25,000 could be regarded as the cost of not having some type of land use control to prevent structures from being erected on land eventually to be used for a highway right-of-way. Other useful types of quantification of aspects of the interchange problem include the information on rezonings that was presented by Professor Horwood and his colleagues, data that Professors Adkins, Pendleton, and others have presented on annual percentage changes of land use in interchange areas (*27*), and information associating development trends with the number of new driveways for certain areas (*2*, p. 70).

Study Approach Needed

One of the useful findings from completed interchange studies is a fuller realization of the complexity of the problem of land usage near an interchange. That problems tend to arise in interchange areas not subject to public control of land planning is a general hypothesis not proved in at least one study (*18*, p. 20). Some interchange areas, where no land use controls were in effect, have been developed without causing any significant traffic congestion; but other interchange areas, where land use controls were in effect, caused land development and traffic problems. Apparently a case study approach—a study in depth—is needed. Then the area of origin and destination for the traffic using an interchange could be delineated. Also helpful would be an explanation of the extent to which land uses a considerable distance from an interchange, such as a hospital, may generate traffic that uses the interchange.

Studies in the detail necessary for an understanding of all aspects of the development of interchange areas apparently will require considerable research effort. Areas having more than one interchange should be selected for study. They could be chosen from broad, overall studies of wide areas, such as aerial surveys, mailed questionnaire surveys, and/or land use inventories, which can be made while a researcher drives through the interchanges. Broad survey studies can also be used for gathering information on secondary but important aspects of the interchange land use problem as to: (1) the optimum distances required for access control along the feeder road; (2) whether orderly development and a sound tax base are associated with adequate land use controls; (3) whether land use of interchange areas becomes specialized—for example, primarily tourist services or community services; (4) how well needs, such as for commuter parking and highway services, are being met; and (5) which interchange areas are developing rapidly, where special land use planning may be required.

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(*26*) *Some Economic Effects of the Suburban Portion of North Central Expressway, Dallas, Texas*, by Russell H. Thompson and William G. Adkins, Bulletin No. 15, Texas Transportation Institute, 1961.

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Interchange and Land Use Control Studies Completed April 1965

Georgia

The Georgia Institute of Technology, in cooperation with the State Highway Department of Georgia and the Bureau of Public Roads, has completed a study entitled *Land Use Planning and Control on the Interstate System in Georgia.* A priority system has been established for counties according to their need for land use planning. Priority 1 includes counties not having a planning commission and in which the Interstate highway has been completed or will be completed soon.

Illinois

Barton-Aschman Associates, in cooperation with the Illinois Department of Public Works and Buildings, Division of Highways, has prepared a study *Highways and Land Use Relationships in Interchange Areas* and supplementary reports. Recommendations are made concerning land use control, design, and location. For land use control, State action is recommended only when the community has not provided control of development in the interchange area. The study report showed that only 15 of 51 counties in which Interstate highways are located had zoning controls and that either existing or potential traffic problems affect 25 percent of the 200 interchanges in Illinois.

Kansas

The League of Kansas Municipalities, in cooperation with the State Highway Commission of Kansas and the Bureau of Public Roads, has completed a study of land use and planning controls entitled *Planning Tools—Theory, Law, and Practice.* This is a study of the theory and application of land use and planning controls available to local governing units for developing safe and efficient streets and highways in the State of Kansas.

Michigan

The Michigan State Highway Department has prepared a report, *Interchange Development Along 180 Miles of I–94,* in which the development near 66 interchanges is analyzed as of 3½ years after the opening of the freeway. The information in the report suggests that the high prices paid by motels and service stations for sites near interchanges had been justified.

Minnesota

Highways—Opportunities and Land Use Controls, a Case Study in Duluth, is a study prepared by the Duluth Department of Research and Planning in cooperation with the Minnesota Department of Highways and the Bureau of Public Roads. The possibilities for coordinating highway planning and local land use planning were investigated and recommendations are made in the report for locations for interchanges, frontage roads, and service facilities such as service stations and motels. Guidelines are also given for use in determining space needed for highway services, and the types of land use controls that might be appropriate for interchange areas are described.

Mississippi

The University of Mississippi, in cooperation with the Mississippi State Highway Department and the Bureau of Public Roads, has completed a report, *A planned Interchange in a Residential Area—Some Interim Influences.* The influence that a planned but uncompleted freeway and a full cloverleaf interchange have on residential property values are considered in this report.

Pennsylvania

Pennsylvania State University, in cooperation with the Pennsylvania Department of Highways and the Bureau of Public Roads, has completed a study, *Planned versus Unregulated Development in a Suburban Community—a Case Study.* In this report, the problems are reviewed that arise in the absence of effective planning in a community where rapid growth is taking place. The policies and programs that local government can utilize to solve growth problems related to highway development are also described.

Tennessee

The Tennessee State Planning Commission, in cooperation with the Housing and Home Finance Agency, has completed a report, *Highway Access Areas in Tennessee.* Suggestions are made on how to plan and guide land development at interchanges by: (1) new legislation to aid local governments within existing planning jurisdictions; and (2) the State initiating a cooperative program in which local government would assume responsibility for areas not in existing planning jurisdictions.

Texas

The Texas A. & M. College, Texas Transportation Institute, in cooperation with the Bureau of Public Roads, has completed a report, *Studies of Land Development at Interchanges.* Recommendations are made that case studies in depth be conducted for interchange areas where the land use problem has definitely been demonstrated.

Virginia

The University of Virginia, Virginia Council of Highway Investigation and Research, in cooperation with the Virginia Department of Highways and the Bureau of Public Roads, has completed Progress Report No. 4 of *A Study of the Economic Effects of the Emporia Interchange, Bypass and Business Loop.* In this report the economic effects on business, land value, and land use in the area of the facilities studied have been analyzed.

The University of Virginia, Virginia Council of Highway Investigation and Research, in cooperation with the Virginia Department of Highways and the Bureau of Public Roads, has also completed a report on *Economic Problems Emerging as a Result of Interchange Patterns on the Interstate Highway System of Virginia.* This report describes the need for considering nonuser effects in justifying highway improvements such as an interchange. The report also presents a model and a problem solution intended to show the relationship between interchange capacity, land use development, land use control devices, and construction and maintenance costs. For purposes of the problem solution by the model, values have been assigned and were not based on experience.

Washington

The University of Washington, Transportation Research Group of the Graduate School in cooperation with the Bureau of Public Roads, has completed Research Reports 21 through 25, *A Study of Land Development Problems at Freeway Interchanges.* The purpose of this study is to identify the land uses competing for sites in approach areas and areas adjacent to highway interchanges, the congestion and traffic-generating characteristics of different land uses, the adequacy of existing controls, and future land needs at freeway approaches and highway interchange areas.

Wisconsin

The University of Wisconsin, in cooperation with the State Highway Commission of Wisconsin and the Bureau of Public Roads, has completed a report, *A Study and Evaluation of Local Highway Planning in Wisconsin.* The report recommends changes in administrative practices and in the scope, detail, and technical content of State and local long-range highway plans. A key is provided in this report to the integration of land use and transportation planning in Wisconsin's rapidly expanding urban areas.

The Department of Resource Development, State of Wisconsin, has completed a study on *The Protection and Development of Interchanges on Wisconsin's State Highway System.* The findings from this study indicate that the Interstate problem should be dealt with in the context of a comprehensive plan based upon economic, population, land use, and other data; the principal elements are land, transportation, and public facilities and services plans.

Other reports

Some State highway departments and planning agencies have issued pamphlets or manuals presenting information intended to facilitate land use planning at interchange areas. These include:

Pennsylvania, *A Manual for Interchange Area Development Planning* and, a pamphlet, *A New Front Door for Your Community.*

Indiana, a pamphlet, *The Traffic Interchange and Community Growth.*

Michigan, a pamphlet, *The New Four Corners—Interchange Areas.*

Ohio, a pamphlet, *Interchange Area Development.*

Subject matter covers such information as: desirable driveway spacing, recommended setbacks, the close relationship between good esthetics and good economics. This information, and a movie on land use at interchanges produced by Michigan, suggests ways in which interchanges can be used for economic betterment.

Interchange and Land Use Control Studies in Progress April 1965

Maryland

A *Freeway Interchange Control Study* has been started in Maryland that is expected to provide information for determining an action program that can be recommended for implementation by the Maryland State Roads Commission in cooperation with the local governments. The aims for this program are: (1) to preserve and improve the capacity and safety of the major highways interchanging with the Baltimore Beltway and other existing freeways, (2) to establish design and land use standards and policies for development of adjoining access highways for other freeways in Maryland, and (3) to evaluate the combined needs for coordinated highway and land development design standards in the interest of highway safety and efficiency, as well as of optimum land development in the vicinity of major highway routes.

Mississippi

Mississippi is conducting a study, *Control of Development of Interchanges.* During the study the best methods will be investigated for controlling development at interchanges in Mississippi's urban and rural areas so that these interchanges can be planned to minimize the effect of business and residential interference on efficient traffic flow.

Nebraska

The Nebraska Department of Roads is in the process of collecting data on land use changes along Interstate 80 between Lincoln and Omaha and is preparing a study on land use near interchanges.

Oklahoma

The University of Oklahoma, Oklahoma Center of Urban and Regional Studies, in cooperation with the Oklahoma Department of Highways is conducting a study of land use patterns near highway and expressway interchanges at selected locations in urban and rural areas in Oklahoma. The purpose of Oklahoma's *Highway Interchange and Land Use Study* is the determination of how interchange location affects land use and how different land use patterns influence the capacity of the interchange.

Pennsylvania

Pennsylvania State University, in cooperation with the Pennsylvania Department of Highways, is conducting a study on *The Impact of Highway Improvement on Land Use, Business Enterprise, and Community Development in Selected Areas of Pennsylvania.* Matters being considered during the study include: procedures for planning and predicting growth at interchange locations, procedures for estimating quantitative measures of the extent to which growth can be explained, and procedures for determining how social deterrents to planning and zoning activities can be overcome. A research model is now being assembled, processed, and analyzed for use in determining the best development plan. An analysis of community receptivity to planning and zoning in 20 or 25 interchange communities along major expressways also will be made.

Utah and Vermont

Utah is conducting a highway interchange and land use control study on *Land Use Adjacent to Interchanges.* Vermont is conducting a study on *Interchange and Land Use,* which includes interchanges on Interstate Highways 89 and 91.

Washington

The Washington Department of Highways is making an inventory of certain highway-oriented businesses at interchanges. Businesses located within 1 mile of the interchanges are to be included in the study.

West Virginia

The University of West Virginia, in cooperation with The State Road Commission of West Virginia, is conducting a study on *Land Use and Planning Controls in West Virginia.*

Aerial Color Photography and Its Use in Materials Surveys

By [1] **JESSE R. CHAVES, Highway Engineer, Aerial Surveys Branch, Office of Engineering and Operations, *BUREAU OF PUBLIC ROADS*; and ROBERT L. SCHUSTER, Department of Civil Engineering, University of Colorado**

Recent developments in the technology of aerial color photography and the advantages and disadvantages of using color film for aerial materials surveys are discussed in this article. The research for this study was undertaken because of the increasing interest in the use of color aerial photography for highway materials surveys—the full potential has not been realized—and the many misconceptions that exist concerning this use of color photography.

The new developments and improvements in the photographic industry are being made so rapidly that even persons currently using aerial color photography have difficulty in keeping abreast of all the technological developments. Therefore, the authors are hopeful that the information presented here will be helpful to those now using this medium and those who may be contemplating the use of color aerial photography for materials surveys.

Introduction

SIGNIFICANT findings in the use of aerial color transparencies in construction materials surveys conducted by the Federal Highway Projects Office, Region 9,[2] U.S. Bureau of Public Roads, are summarized in this article. Selected data and costs of aerial films and processing thereof, as well as trends in use and development of aerial color films, are presented, and suggestions are made as to areas in which more research is needed.

The evaluation of aerial color photographs, reported in this article and a previous one (*1*),[3] was a secondary objective of a comprehensive survey of construction materials. The total project was not designed as a controlled research experiment, and the results are representative of fairly restrictive climatic conditions, geographic area, and different geologic materials.

Photographs were taken along the major road system and in other selected locations in Yellowstone National Park, Wyoming. Aerial color transparencies (Kodak Ektachrome Aero Film, replaced in 1962 by an improved product called Ektachrome Aero Film, Process E–3) were made at a scale of 1:6,000 (500 feet to 1 inch) in single flight strips. The photographs were taken with a 6-inch focal length Pleogon lens from an average flight height of 3,000 feet. More than 400 flight-strip-miles were photographed on color film during the summers of 1961 and 1962. Photographs of selected segments, totaling about 100 linear miles, were taken with conventional panchromatic film at the same scale.

The color transparencies were examined in three dimensions by using a mirror stereoscope and portable light source. Potential construction material sources thus located were annotated on transparent plastic envelopes containing the aerial photographs; then, each source was verified by ground inspection. Potential sources that appeared to have the best construction material were further investigated in the field with a backhoe, after administrative approval to examine each potential site had been obtained. Representative samples were taken from each source investigated and laboratory soil tests were performed for engineering evaluation.

Findings

As a primary result of the Yellowstone Park study and other materials surveys in which aerial color photography was used in Region 9, it is believed color photography has many advantages over black-and-white photography for use in materials surveys. Some of these advantages have been previously reported (*1*).

The use of panchromatic aerial photographs for interpretation of specific ground conditions, soils, or geologic materials involves the interpretation of different photographic tones. The number of such tones or shades of gray that can be differentiated is extremely limited, and many different types of soils or geologic materials may have about the same tonal expression. As the human eye can perceive about 20,000 shades and hues of color, interpretation of color photographs is considerably easier than that of black-and-white photographs. Identification of materials deposits can be based partially on the color of the deposits rather than having to rely on photographic tones of the panchromatic photographs. The authors realize, of course, that neither color nor photographic tone alone can be used in identifying materials, but they do represent one of the more important elements used in photographic interpretation.

An extreme example illustrating the shortcomings of black-and-white photographic tones for use in photographic interpretation was noted in the Yellowstone Park study. Occasionally it was extremely difficult to differentiate between certain low-growing types of vegetation and soil or rock on black-and-white photographs because they produced identical gray photographic tones. Differentiation was, of course, very simple on the color transparencies. This is an extreme example of the difficulty that can arise in use of panchromatic film, but it was paralleled by the difficulty encountered in attempts to differentiate between types of materials.

Minard and Owens (*2*) in mapping soils and geology of the Atlantic Coastal Plain in New Jersey noted that certain rock and mineral types can be identified much more readily from aerial color photographs than from panchromatic photographs. This was also true in the Yellowstone Park study where aerial color photography was used. Color differences of sands and gravels that contained large percentages of obsidian or siliceous sinter could be differentiated from sands and gravels composed of the more common rock assemblages. The distinction between these different sands

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[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., Jan. 1964.

[2] Includes Colorado, New Mexico, Utah, and Wyoming.

[3] References indicated by italic numbers in parentheses are listed on p. 179.

and gravels was not nearly so obvious on panchromatic photographs of the same areas.

Color differentiation also has proved to be superior to black-and-white photographic tones for identification of engineering materials and geologic features on other projects undertaken in Region 9. For example, in Dinosaur National Monument, Colorado and Utah, where geologic formations are well exposed and of variegated coloring, a comparison of color positive transparencies with black-and-white prints at a scale of 1:6,000 showed that: (1) differentiating between strata of sandstone and shale is easier on color than on panchromatic photographs, (2) flat erosional remnants of granular mountain outwash that overlay dipping sedimentary formations are more easily and more positively identified on color than on panchromatic photographs, (3) highly fractured light-colored limestone strata can be identified and differentiated about equally well on either type of photographs, and (4) river terrace remnants can be more reliably identified on color photographs.

Organic soils, wet soils, boggy ground, and seepage zones also can be more readily identified on color photographs than on panchromatic because of their distinctive greens and browns. Minor drainageways, poorly drained depressions and swales, and seepage areas in landslides show up very clearly on color photographs. On panchromatic photographs such areas are displayed in generally darker photographic tones, but identification or delineation of these areas cannot be as positive; sometimes the conditions are not apparent even to an experienced interpreter.

Different vegetative types can also be recognized more readily on color than on panchromatic photographs; correlation can be made between type of vegetation and specific types of materials or ground conditions. Thus for areas covered by vegetation, the color photography is superior to panchromatic. Image identification is easier on color aerial transparencies than on black-and-white prints because of the color contrasts provided by the transparencies. Color transparencies are especially helpful in making identification of cultural features such as trails and aerial targets.

Although aerial color photography generally is superior to aerial panchromatic photography in terms of ease of interpretation and quality of results obtained, it does have disadvantages. The most obvious of these disadvantages is cost. Generally, aerial color photography costs approximately three to four times as much as the conventional panchromatic photography. This cost comparison reflects only the actual cost of the film material and its processing. The cost factor is discussed in greater detail later in this article.

Another disadvantage of using color photography is the difficulty encountered in obtaining proper film exposure, especially in areas where extreme lighting contrasts occur within the area covered in a single photograph. For example, in the Yellowstone Park study, hot springs deposits that were light in color and devoid of vegetation provided such sharp contrast to the surrounding dark vegetation that their color often exceeded the exposure latitude of the color film. For such light-colored areas, any colors present were often washed out in the photograph, and images generally were not well defined.

In the Yellowstone Park study considerable differences in quality of color reproduction were sometimes noted between flight strips of the same areas photographed at different times. For example, some overlapping flight strips of aerial color photographs taken during the summer of 1962 had generally better color reproduction than those taken in 1961. This improvement was particularly noticeable for areas of dense vegetation where green hues were predominant; the 1962 photography generally reproduced the other areas and different colors of the features in a more natural way than the 1961 photography. The more natural color reproduction was particularly helpful in mapping old, gray, sandy beach deposits in heavily forested areas. In the 1961 photography, the gray colors were mostly obscured by an overall greenish cast of the photographs; this made the detection of the beach deposits difficult. The improvement in the films made in 1962 was attributed to more accurate film exposure.

Another fault of color transparencies as compared to panchromatic photographs is the relatively expensive production of duplicate transparencies and the difficulty of obtaining color reproduction of the originals: considerable experience is required for this process. Therefore reproduction of color transparencies is rather impracticable; if a color transparency is lost or damaged, it cannot be replaced as can a black-and-white print. When one is working with color transparencies, considerable care in handling must be exercised.

Because color transparencies must be viewed by transmitted light, a suitable illuminating source is necessary if they are to be used successfully for materials surveys. This has not generally handicapped office use of the transparencies, but it can prove to be somewhat of a problem when the transparencies are used in the field. In the Yellowstone Park study, special portable light boxes were used. They were provided with a vibrator power supply that could be connected to an automobile electrical system. The use of these portable light boxes was fairly successful, but it was awkward to work with them in the field vehicle and, of course, they could not be used away from the vehicle.

The necessity for transmitted light when color transparencies are viewed also means that a mirror stereoscope must be used; individual transparencies cannot be easily overlapped, as can black-and-white prints. In the field, a mirror steroscope is relatively more awkward than the smaller lens stereoscope commonly used for viewing black-and-white prints.

As noted previously, in the Yellowstone Park study, primarily aerial color transparencies were used, and the comparisons made here have been between black-and-white photographs and these aerial transparencies. If color prints were to be compared with black-and-white prints, it is obvious that some of the findings and conclusions from the Yellowstone Park study would no longer be valid. For example, transmitted light would not be needed for viewing the color prints; and adequate duplicates of color originals would be available, thus eliminating the danger of loss or damage to the originals during interpretive work.

In one area of Yellowstone Park some short flight strips were photographed with Agfacolor Negative Film CN 17. Because these photographs were taken in conjunction with another project, only cursory examination was made. Contact color prints tended to have an excess of green, but they had good resolution. It was later determined that these photographs had been printed with a light source that did not provide a full spectrum. Contact prints subsequently were made with an appropriate printing light source and the color balance was greatly improved. Black-and-white prints made from the same color film negative were of excellent quality.

Procurement of Aerial Color Photographs

Several of the more important aspects in the procurement of aerial color photographs are discussed in the following paragraphs.

The distance between the project to be photographed and the airfield at which the photographic aircraft is based is extremely important in either a black-and-white or color photographic mission. Some delay in procuring color photography for the Yellowstone Park study was experienced because the photographic aircraft and crew were based several hundred miles from the project area and could not take advantage of short periods of excellent photographic weather during prolonged cloudy and rainy periods.

Because the exposure latitude of most color film is narrower than for conventional black-and-white film, exposure difficulties are encountered where there is extreme contrast in the light reflectance within an area photographed. The light-colored areas tend to be overexposed. Consequently, colors wash out and images are not always registered distinctly. In flying over areas that are either light or dark, the photographer can properly adjust the exposure for the given scene. In this connection it is well to note that the ability of the photographer to think and use good judgment in making photographs under varying conditions of light, haze, clouds, and changing subject matter is important in procuring quality color photographs.

As flight heights increase, particularly in certain geographic areas and at certain times of the year, haze tends to cause color photographs to have an overall bluish cast and thus subdues original ground colors recorded on the film. The effects of haze are less noticeable for relatively large-scale photographs taken at fairly low flight heights. Different color films also have greater or lesser sensitivities to haze. Haze filters and compensation in the printing process can be used to minimize the effects of

haze. To overcome some of the haze difficulty, those making aerial color photographs are tending to use high quality, shorter-focal-length lenses, which permit lower flight heights for a given photographic scale. In addition to reducing the haze problem, such high quality, short-focal-length lenses sharply reduce the amount of darkening of corners of photographs, a common problem related to color aerial photographs taken with inferior lenses. Antivignetting filters on the aerial cameras and automatic-dodging printers also can minimize corner darkening.

Filters used with an aerial camera reduce the amount of light reaching the film and sometimes are a source of difficulty. For example, in some of the flight strips made in the Yellowstone Park study, the images in the corners of the aerial photographs were blurred. It was later determined that this blurring was caused by a faulty color-balancing filter that had inadvertently been exposed to excessive heat. Fortunately, the blurring was not sufficiently serious to affect the use of the photographs.

Color balance of an aerial photograph depends upon the geographic latitude, season of the year, time of day, amount of haze and cloud cover, film exposure, type and variation of emulsion, type of filters used, and film processing and printing. One need not wonder why the attainment of true color reproduction in an aerial photograph is a very difficult task. However, the excellent results that can be obtained despite all these deleterious factors are remarkable.

Trends in Aerial Color Films

Through continued research by film manufacturers, improvements in aerial color films have steadily been made. Many former objections and criticisms of color films are no longer valid. Significant trends, particularly within the last 5 years or so, have been: (1) a large increase in emulsion speed or light sensitivity, (2) reduction in granularity and consequent increase in resolution, (3) improvements in dye formulas that provide a stronger degree of color fidelity, (4) wider latitude of exposure, and (5) decreasing costs.

Standards of sensitivities and resolution have been reached today that were hardly thought attainable by many only 10 years ago. Costs of color film have been decreasing, and this trend is expected to continue as aerial color film comes into more general use. One Government agency has reported that the cost of color photographic paper has been reduced by more than 50 percent in the last 3 years, mostly because of the increase in demand.

Aerial color negative films of both domestic and foreign manufacture have recently been introduced in the United States. The advantages of aerial color negative film over color positive transparencies make it appear that negative film will come into greater use, particularly as improvements are made in emulsion speed and exposure latitude, and as costs decrease. Use of color negatives provides a greater flexibility as both black-and-white prints and color prints can be made from the same negatives. The negatives can be saved and additional prints made as required. Compensation for incorrect exposure, haze, vignetting, and color balance can all be made in the photographic laboratory. Color-balancing filters are not required on the aerial camera as they are when reversal-type film is used.

Generally the paper base material on which color photographs are printed has many of the same characteristics as the base material used to print black-and-white photographs. At least one film company produces color print material that is tough, waterproof, and dimensionally stable (*3*). The office and field problems associated with use and storage of paper-base prints are well known.

Summary of Some Aerial Color Film Data and Characteristics

The following listed aerial color films are now commercially available. Only selected characteristics are included. Further information regarding these films can be obtained from the respective manufacturers.

Kodak Ektachrome Aero Film, Process E–3, is a color reversal film from which positive transparencies are obtained. This film replaces Kodak Ektachrome Aero Film, High Contrast. This film has an Aerial Exposure Index of 25 that is about three times as fast as the previous film that had a speed index of ASA 40 daylight. Haze filters are recommended and color-balance filters are used when necessary for particular emulsions. The Aerial Exposure Index is a number assigned by the Eastman Kodak Co. to indicate the speed of film. It is used in calculating film exposure with Kodak Aerial Exposure Computer and should not be confused with ASA Exposure Index (*4*).

Kodak Ektachrome Infrared Aero Film, Process E–3, replaces Kodak Ektachrome Aero Film (Camouflage Detection). The new film is a false-color, reversal film that has three layers sensitive to green, red, and infrared radiation, rather than blue, green, and red as in conventional color film. A yellow Kodak Wratten Filter No. 12 is used to absorb blue radiation to which all three layers are sensitive. Color compensation filters may be used for color improvement of transparencies. This film has an Aerial Exposure Index of 10, which takes into account the use of the yellow filter.

Kodak Ektacolor Professional Film, Type S, is color negative film that has recently become available; it has a speed index of ASA 80 daylight. Use of a haze filter is recommended. Color balancing is accomplished in the printing process. (A recent experimental trial by the authors showed that good-quality aerial color prints can be produced from this negative film. This film appears to have reasonably wide exposure latitude without objectionable shifts in color balance. Although not designed for aerial photography, it has fairly high contrast and good haze penetration without the use of a haze filter.)

Ultra-Speed Anscochrome is a color-reversal film from which positive transparencies are obtained. The speed index for this film is ASA 200 daylight. By forced processing, the speed can be pushed to ASA 400–800. This film has a temperature-color balance of 6,000° K. and has high resolution when exposed under normal conditions. The latitude of exposure is about plus or minus one-half lens stop for accurate reproduction and good color saturation. Color positive prints (Printon) can be made from this film.

Super Anscochrome is a color-reversal film having a speed index of ASA 100 daylight. Exposure latitude and color balance are similar to Ultra-Speed Anscochrome. It has slightly higher resolution and provides somewhat less contrast than the latter.

Agfacolor Negative Film CN17 is a negative color film having a speed index of ASA 40 daylight. Typical exposures are 1/250 to 1/450 second at f4 to f5.6 under average conditions of brightness. Satisfactory results can be obtained at one to two lens stops lower than for optimum exposure. Ultraviolet and light yellow filters can be used. Color balancing is accomplished in the printing process.

Since this article was originally written, several new films have become commercially available. They are: Ektachrome Infrared Aero, Type 8443; Ektachrome Aero, Type 8442; Special Ektacolor Aerial, Type SO–282; and Anscochrome FPC–132 and Anscochrome FPC–289. Information regarding these films may be obtained from the manufacturers: Eastman Kodak Co., Rochester, N.Y., and Ansco, Binghamton, N.Y.

Cost of Aerial Color Photography

One of the foremost considerations to most persons, and a question frequently asked is, "How much does aerial color photography cost?" Perhaps a followup to this question should be, "Is aerial color photography worth the cost?" To answer the latter question first, the authors are of the opinion that the answer in most instances is an unqualified yes. Perhaps one could apply the saying, "You get what you pay for," with respect to aerial color photography.

No attempt has been made to make a cost analysis here, because no matter how completely one analyzes the economics of aerial film, the question remains unanswered as to whether color aerial film is worth its cost. As previously mentioned, raw aerial color film and processing costs are about three to four times the cost of conventional black-and-white photography. This is not surprising, as color films have three emulsions and panchromatic films have only one. The price of chemical kits used for prosessing both types of film is about the same.

In terms of the entire operation, including the photographic mission itself, the cost of procuring and processing aerial color transparencies is only about twice that for panchromatic film. Sometimes both types of photographs may be procured during the same mission. Other factors influencing cost are the total number of photographs required, length and number of changes in direction of

flight strips, and distance of the project from the base of operations.

However, one government agency, which takes its own photographs and does its own photographic laboratory work, reports that the cost of producing color prints is about four times that for black-and-white prints.

One commercial aerial color processing photographic firm quotes a price of $90 for processing negative color film (a roll 9½ inches by 100 feet) and $115 for color-reversal film of the same dimensions. Color paper prints that are not color balanced can be obtained for about $0.90 a print. Prints that have been color corrected and dodged for lens vignetting cost $3 each.

A price quotation obtained for 1:6,000-scale photography from a commercial photogrammetric firm was $55 per flight-strip-mile. This price included one set of contact color prints, one set of black-and-white prints, and one photographic index. Additional color prints could be obtained for $15 per flight-strip-mile, and additional black-and-white prints could be obtained for $1 each.

Aspects of Color Film Processing and Printing

Only a few of the most important aspects of aerial color film processing and printing are presented here. No attempt is made to discuss all the ramifications of this complex subject.

One of the more important phases, and one that is often neglected, is the extreme care necessary in the photographic laboratory in regard to general cleanliness and prevention of contamination of chemical solutions. Good housekeeping is a must in a color photography laboratory. In addition, processing procedures must be rigidly controlled. The general attitudes, habits, and procedures followed in most black-and-white photographic laboratories are not good enough.

One phase of color processing most misunderstood is the time required in the photographic laboratory. Usually, the total time required to process a role of exposed aerial color film is only slightly longer than that required for panchromatic photographs, if the proper equipment is available. Processing of color-reversal film takes slightly longer than for color negative film. Although color processing time does not differ significantly from the time for panchromatic, the complete attention of the laboratory technician is required throughout the color-processing operation. This full-time attention is not required for processing panchromatic photographs.

Two government agencies report that their production capacity for panchromatic prints is from 2 to 2½ times greater than that for color prints. This comparison illustrates the additional time required to make color prints with the equipment used in these particular photographic laboratories. Most laboratories use the so-called wind-and-rewind type of aerial color film processing equipment. Continuous processing such as is used in the motion-picture industry is not made to accommodate aerial color film. Continuous processing equipment is very expensive, and at the present time cannot be justified by the quantity of aerial color film processed.

Contact printers, either conventional or automatic-dodging type, equipped with full spectrum tubes, can be used to make color prints from either positive transparencies or color-film negatives. The color balance of each photograph, if required, can be regulated by using special filters or adjusting the printing light source. Compensation for haze can also be made in the printing process by removing some of the blue color with filters. Differences in film exposure and light quality (time of day) can also be compensated for in printing. By using automatic-dodging printers, a more uniform density can be obtained throughout a given photograph, and the photographs in a flight strip will tend to be more nearly uniform. Some objection has been voiced regarding the use of automatic-dodging printers because serious color-balance shifts occur on some films; this shift is due to reciprocity failure (longer exposure time required in printing). Attainment of perfect color balance in each print remains a costly and time-consuming process.

Figure 1 shows the possible products (contact size) that may be obtained from color negative and reversal-type films. Enlargements can also be made from both types of film.

Viewing Aerial Color Photographs

A few special requirements must be met for optimum stereoscopic viewing of aerial color photographs. For optimum color perception, a light source that radiates energy over the entire color spectrum is necessary. Some modern fluorescent lights and combinations of fluorescent and incandescent lights meet this requirement. Most of the light sources used for photographic interpretation of aerial color photography in the past have been improvised. Recently, special fluorescent tubes that are coated with selected phosphorus and provide energy in the entire color spectrum have become available. This light source is color balanced at 3,900° K. and has intensities of 400 and 600 footlamberts. The higher intensity is used for viewing dense transparencies. Diffusers provide an even distribution of light. This special light source has the added advantage of maintaining its color balance and intensity of output for long periods of time, something ordinary fluorescent light cannot do. As is true for other fluorescent light sources, providing for heat dissipation is not the problem it is when incandescent bulbs are used.

Double projection photogrammetric instruments currently in use have not been designed for use on aerial color photographs. Use of color aerial photography in these instruments, both qualitatively and quantitatively, has been experimental in nature up to this time. In the Kelsh stereoscopic plotter, color positive transparencies sandwiched between glass plates have been used by means of polarized light and a special platen that has a surface of anodized aluminum. Despite its limitations, this approach appears to have considerable merit, particularly for intepretation purposes (*5*). Color transparencies have also been experimentally employed in optical-train photogrammetric instruments. As these instruments do not depend on the anaglyphic principle for their operation, polarized light is not required for stereoscopic viewing. This approach has proved successful for qualitative use.

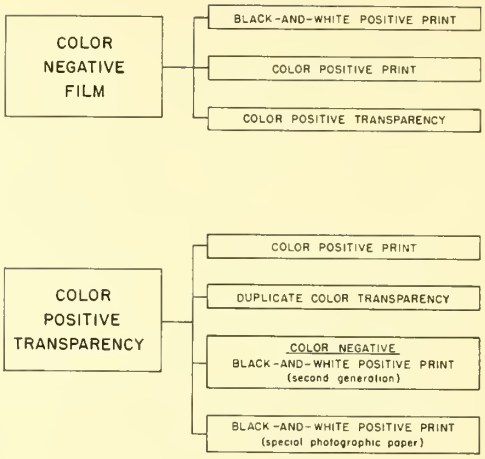

Figure 1.—Possible products from color negative and reversal-type film.

Color diapositive plates for use in photogrammetric plotting instruments are now commercially available (Kodak Special Plates, Type 032–01). Diapositive plates eliminate the dimensional instability problem associated with positive film transparencies. The color diapositive consists of duplicating film laminated on a 0.250-inch glass plate. As a result of this recent development, more extensive use of color photographs will be possible in photogrammetric instruments, both quantitatively and qualitatively.

Although not specifically designed for the 9- by 9-inch size format used extensively in highway work, a stereoscopic viewing instrument of interest to the military services and others has recently been developed. This instrument permits the viewing of projected 70-mm. strip color and conventional photographs in three dimensions without the aid of polarized light or colored lenses. This is accomplished by a special projection system and viewing lens. Magnification is also provided.

Quality of Color Reproduction and Photographic Interpretation

Establishment of a set of specific rigid criteria as a basis for evaluating the acceptability or nonacceptability of aerial color photographs for use in materials surveys is nearly impossible and at best not practical. General requirements applying to panchromatic photographs procured for interpretive

purposes are applicable to color photographs. The judgment of the particular user must be the prime basis for governing decisions regarding acceptability of color photographs for any given project.

To the layman, as well as to the trained interpreter, natural colors accurately reproduced in an aerial photograph are more pleasing than poorly reproduced colors. The value of true color reproduction, however, in interpretation of soils, locating sources of construction materials, and mapping geologic formations is questionable when considered in terms of the effort and cost of attaining this objective.

Although true color reproduction would probably be esthetically desirable to obtain, the effort and cost are at present too great to warrant it. Actually, color distinctions and differences are of paramount importance, and it may sometimes be necessary to disturb good color balance to obtain certain color contrasts.

Proper film exposure enhances color differences and is particularly important when color-reversal film is used, as there is little leeway in the photographic laboratory to compensate for errors. Overexposure generally tends to wash out colors. Latitude of exposure varies considerably with different color films and for some films it may be desirable to underexpose rather than overexpose. The reverse may also be true for other films. For critical work, test exposures should be made with the same film (same emulsion number) that is to be used on the project. Laboratory procedures should be standardized.

Resolution in an aerial photograph is an important consideration. It may, however, be less important on a color than on a panchromatic photograph because color rendition and contrasts are probably more important in differentiating images. Aerial color films generally have much greater contrast than color films produced for ground photography. Aerial color photographs having high image resolution are certainly desirable, but other considerations may be more important for photographic interpretation. When photographs are to be viewed under magnification or when enlargements are to be made, resolution becomes more important.

Research Needs

The body of available knowledge related to use of aerial color photographs for interpretive purposes is still relatively small. As most State highway departments do not have the time, trained specialists, facilities, or funds to undertake comprehensive research projects of a basic nature, most research in this area has been conducted by the universities and by agencies of the Federal Government. Though considerable research in this field has been conducted by the military services, the results of this work generally have not been made available to civilian engineers and scientists because of security restrictions.

The need for research in the use of aerial color films for materials surveys and for interpretation in general cannot be denied. The authors have suggested some avenues of possible research activity.

The selection of the proper film and/or filter for use on specific projects having unique color combinations needs to be investigated. Some research of this type has been done more or less on a trial-and-error basis in which different film-filter combinations and film exposures have been used. A limited amount of work has been done through an approach that eliminates most of the guesswork, by using spectral reflectance measurements (spectrophotometric studies) of the soils and rocks to be photographed. The results of these measurements show the dominant wavelengths of reflected light and enable photographers to select the appropriate filter so that optimum color contrasts will be shown (*6*, *7*).

Aerial photographic interpretation has in the past been largely qualitative and subjective in nature. Although the amount of information that can thus be obtained is very large, the quantitative approach to interpretation may possibly be the approach that will really be effective. This approach has long been neglected and is in need of further study. By use of a densitometer or microdensitometer, color transparencies or prints can be scanned and the intensity of specific wavelengths of light, either transmitted (transparencies) or reflected (prints), can be measured and recorded. Normally, measurements of the amounts of red, blue, and green light are made. Particular rock types, soils, and landforms can then be identified and differentiated. Distinctions among geologic formations and soils often can be made in this manner that cannot be made visually (*7*, *8*, *9*).

Experimentation is needed with all types of aerial color film to determine the latitude of film exposure in relation to the value of the resultant photographs for interpretive purposes (*10*). Research is needed involving the use of different combinations of light sources, reflectors, filters, and light intensities in color-viewing systems for interpretation. This phase of enhancing color differences for interpretation has been neglected. The emphasis has been placed on the taking and processing phases of color photography.

The potential of false-color films for interpretive purposes needs to be evaluated. False-color films differ from ordinary color films in that one of the three emulsion layers is sensitive to infrared. Colors produced by this film after processing are not those of the photographed objects such as soil or rock. Color contrasts and differences depend to a large extent on the differences in infrared reflectivities of the scene photographed. Therefore, use of false-color film is more advantageous than use of conventional infrared film, in which the record is in shades of gray (*4*, *11*).

To ascertain the degree of stability of dyes under ideal storage conditions and under routine office and field use, controlled experiments in aerial color film deterioration are needed. The effect of certain chemicals used to retard the bleaching of colors on color prints should be ascertained. Normally, color prints are soaked in these solutions. When dried, the then invisible coating reduces the penetration of ultraviolet rays, which bleach dyes. Particular types of vegetation have been correlated with specific soils and ground conditions and can be used by trained interpreters to identify these conditions on aerial photographs. Recognition of vegetative types is unquestionably greatly facilitated by use of aerial color photographs. Research is needed to correlate identifiable vegetation on aerial photographs with specific conditions and materials on the ground. False-color films may have some application and potential use. This approach to interpretation probably will have its greatest success in semiarid and subarctic regions.

More test flights and use of aerial color photographs are needed over a broader area of climate, topography, and geologic materials. Perhaps a designed experiment in which the test areas would represent many conditions would be appropriate. Worthwhile results probably could come from experiments involving special photographic laboratory procedures to generally improve color photography for interpretation.

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(References on page 179)

Effectiveness of Sign Background Reflectorization

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BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

This article describes a study conducted to compare the brightnesses of signs having different degrees of background reflectorization and to analyze their effect on the ability of drivers to follow a test route to a given destination on controlled-access highways in a suburban area. The results of previous studies by the Bureau of Public Roads have indicated little practical difference in legibility of signs having different degrees of background reflectorization, including no reflectorization. But it was thought possible that the degree of background brightness might affect other sign-effectiveness factors such as detectability or reading time. Data have been compiled on the performance of a total of 150 volunteers, who attempted to follow TEST ROAD signs leading to 18 exit turns. Fifty of these volunteers were tested on each of three successive nights. One-third of the turns were marked with signs having each of three different background materials. The frequency of errors by materials was not statistically significant. The methodology used in this study may have applicability in related work.

Introduction

TO DETERMINE whether reflectorized backgrounds contribute to the overall effectiveness of highway signing, the Bureau of Public Roads conducted a study in which signs having backgrounds of different degrees of reflectorization were compared. This comparison was accomplished by analyzing whether the degree of background reflectorization seemed to affect the ability of test drivers to follow signs at night and to negotiate a test route to a given destination on controlled-access highways in a suburban area.

Present standards for directional signs on the National System of Interstate and Defense Highways require white legends on a green background. If the sign is not self-illuminated, the legend must be reflectorized; however, for roadside signs and illuminated overhead signs, reflectorization of the background is optional. There has been some question as to the degree of background brightness that is necessary and, consequently, the degree of reflectorization. The results of previous studies by the Bureau of Public Roads have indicated little practical difference in legibility of signs having different degrees of background reflectorization, including no reflectorization.[1] It is possible, however, that the degree of background brightness affects the effectiveness of other sign factors such as detectability or reading time.

Description of Study

Test subjects drove over a route that included highways and parkways of the Pentagon network and other controlled-access highways in suburban Virginia near Washington, D.C. Changes in the direction of the route were marked by test signs placed in advance of the exits. An observer riding in the car with the test subject noted missed turns and directed the test drivers back onto the route when a turn was missed. To conceal the true nature of the experiment, the test subjects were told that they were participating in a study of ". . . night driving characteristics . . ." and that all of them were being asked to use the same route so that the same base could be used for comparison of results.

The test route illustrated in figure 1 was approximately 25 miles long; its controlled-access design, in which the alinement was somewhat curvilinear, dated back to the World War II period. All 18 of the test turns required right exits. Each was marked by a single test sign (fig. 2) placed 400 or more feet before the exit. All parts of the route on approaches to and in the vicinity of test signs and turn exits were on controlled-access, divided highways having 2 or 3 lanes in each direction. Some exit ramps led to streets or highways of lesser design, but the route always led back to an access-controlled, divided highway, usually one-half mile or more before another test sign. Posted speed limits on the route ranged from 35 to 55 m.p.h., but were mostly 35 or 40 m.p.h. Prevailing speeds on the same highways ranged, in most places, from 35 to 45 m.p.h. All test turns on the route were marked with test signs having reflectorized white legends and green backgrounds.

Sign backgrounds

Three degrees of reflectorization were used for the test sign backgrounds: (I) a nonreflectorized green background; (II) a moderately bright reflectorized green background; and (III) a relatively high-brightness reflectorized green background (standard reflectorized sheeting). Specific luminance curves for divergence angles up to 1 degree at an incidence angle of $\frac{1}{3}$ degree for these materials and for the material used for the white legend are shown in figure 3. For comparison, the specific luminance of unity of a theoretical, perfectly white, diffusing surface is included. The nighttime appearance of the different materials at one location is illustrated in figure 4. Because of the limited range of brightness that can be depicted by photographic process, the views are only approximate representations

The type of background at each turn was assigned at random each night. The only limitations on the assignment were that each night the sign marking each turn had to have a different background and one-third of the turns each night had to have had each of the three background treatments. Therefore, six turns each night were indicated by signs having each of the background treatments. On successive nights, the sign background was changed at each test turn. Each of the 18 turns was therefore marked on successive nights by a sign having each of the background types. Three nights of testing (or multiples of three) were needed to balance the experimental design of background treatments and turns. A schedule of backgrounds by nights and turns is shown in table I.

[1] Information was gathered as part of the Interstate sign tests conducted by the Bureau of Public Roads at Riverdale, Md., Nov. 1957.

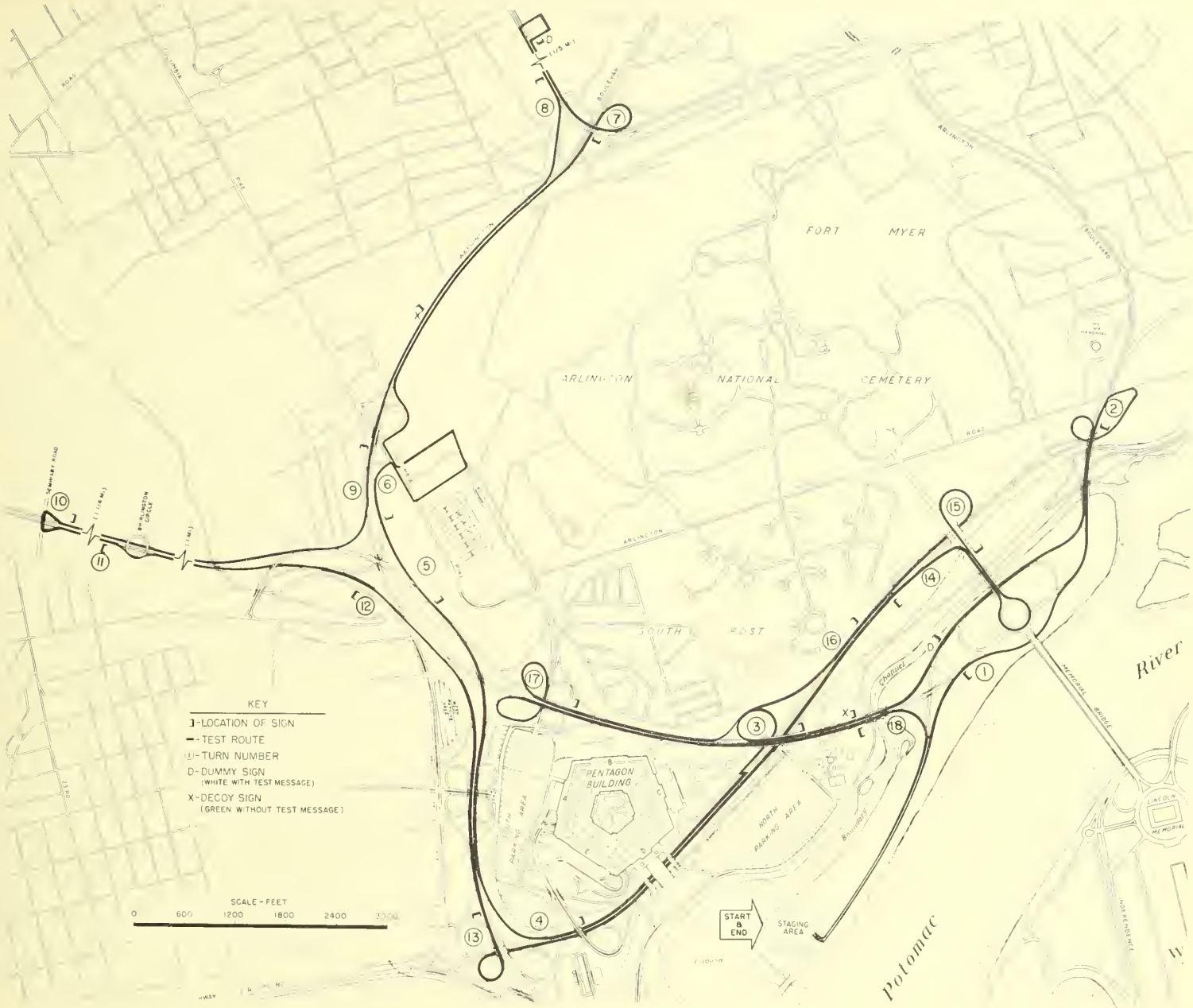

Figure 1.—Map of area in Washington, D.C., and suburban Virginia showing test route.

To facilitate changing the test sign backgrounds on successive nights, three signs were fabricated for each test turn; they were identical in legend and legend spacing and differed only in background treatment. Legends and borders were white reflectorized sheeting. Eight-inch series D letters were used. The sign carried two lines of copy and appropriate arrows: The first line was a legitimate destination on the route, for example *PENTAGON;* the second line was always *TEST ROAD.* All of the test signs measured 96 by 32 inches. They slid into horizontal metal channels mounted on posts, at a height measuring 5 feet from the bottom of the sign to the pavement. The posts were a minimum distance of 2 feet beyond an unmountable curb or edge of a shoulder. The signs were mounted at approximately right angles to the roadway, but the angle always exceeded 90 degrees in order to minimize specular reflections.

Precautions

The test signs were not conspicuously different in size and legend style (letters and arrows) from the permanent directional signs along the route, most of which had either black legends on a white reflectorized background or white reflectorized legends on a nonreflectorized background. None of the test signs gave directions that conflicted with or appeared to duplicate the permanent signing. In a few places, a test sign temporarily replaced a permanent sign.

Even with these precautions, the subjects could have learned to associate, either consciously or subconsciously, the test message with signs having green backgrounds. This would have biased the results in favor of reflectorized backgrounds. Furthermore, subjects could also have learned to associate the test message with the low, wide, rectangular shape or the two-line legend and arrow design of the test signs. So, to further camouflage the differences between the test signs and the permanent signs, two dummy signs that more nearly resembled the permanent signs were placed along the route. Dimensions and color combinations—black legend on a white reflectorized background—of the dummy signs were similar to the permanent signs. In addition, two decoy signs were placed. The dummy signs are illustrated in figure 5 and

Table 1.—Schedule of sign backgrounds, by nights and turns

Turn number	Sign background material [1]		
	1st night	2d night	3d night
1	I	II	III
2	I	III	II
3	III	I	II
4	III	II	I
5	I	II	III
6	II	I	III
7	III	I	II
8	I	III	II
9	I	II	III
10	II	III	I
11	II	III	I
12	II	I	III
13	III	II	I
14	III	I	II
15	I	II	III
16	II	III	I
17	II	III	I
18	III	I	II

[1] Roman numerals refer to degree of reflectorization as follows: I, nonreflectorized green background; II, moderately bright reflectorized green background; and III, relatively high-brightness reflectorized green background (standard reflectorized sheeting).

Figure 2.—Day view of test sign.

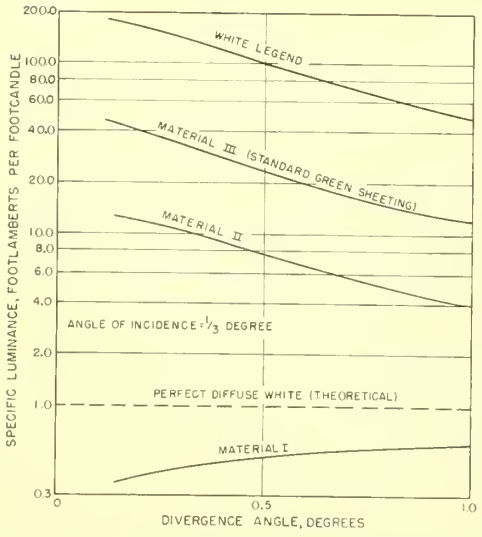

Figure 3.—Specific luminances of materials used on test signs for an incidence angle of 1/3 degree and divergence angles up to 1 degree.

Table 2.—Characteristics of test subjects

Characteristic	Number of test drivers—		
	1st night	2d night	3d night
Organization: [1]			
Bureau of Public Roads	20	20	20
Sports Car Club	23	19	31
Jayeees	9	9	7
Others	12	15	6
Sex of driver: [2]			
Male	47	45	43
Female	3	5	7
Age of driver (years): [2]			
Under 20	2	1	0
20–29	18	13	20
30–39	14	10	15
40–49	7	16	6
50–59	8	7	6
60 and older	1	3	3
Visual acuity:			
Below 20/40	50	49	49
Above 20/40	0	1	1
Familiarity with area: [3]			
Very familiar	10	11	8
Moderately familiar	24	28	25
Unfamiliar	16	11	17
Average age of driver	35.1	39.3	35.9
Median age of driver	33	40	33

[1] Estimated from mail return and schedule of volunteers. Since subjects were identified by code number at site, there is no way of identifying the 50 subjects who participated each night by organization.
[2] Taken from 1st questionnaire.
[3] Taken from 2d questionnaire. Subjects queried after completing course.

the decoy signs in figure 6. The decoy signs resembled the test signs in shape, design, and color combination, but the test message was omitted. Supplemented by the few permanent signs having green backgrounds, the decoy signs were intended to prevent the subjects from associating green backgrounds with only the test signs. Small 24- by 30-inch auxiliary signs used for directions on ramps and other connections on the route also had black legends on white reflectorized backgrounds (fig. 7).

Procedures

Test subjects were volunteers from organizations such as a local Junior Chamber of Commerce, a local Sports Car Club, the Bureau of Public Roads (other than the Traffic Systems Division), and other agencies of the Department of Commerce. Subjects were scheduled in advance for specific nights and times of arrival at the staging area.

The test procedure was essentially the same as that used in a previous study of route-turn markers.[2] On arrival at the staging area, subjects received a printed sheet that contained explanations and instructions. They were asked to complete a questionnaire on driving experience and to undergo a vision test on a Keystone Visual Safety Tester for acuity, stereopsis, and far-point fusion. They were also tested for color vision. The questionnaire and vision test were part of the general study of night driving characteristics.

[2] *Advance Route-Turn Markers on City Streets*, Lawrence D. Powers, PUBLIC ROADS, vol. 32, No. 1, April 1962, pp. 12–16.

Figure 4.—Night views of signs at same location having different background materials. Brightnesses are approximate. Background materials I, II, III were on test signs from top to bottom, respectively.

In addition, tests served in screening subjects for visual defects. The questionnaire and visual acuity data are summarized in table 2 for the different groups of subjects taking part in the study each night.

To retain anonymity, subjects were identified by code numbers. It was explained that the observer would usually be too busy to give directions, and that the drivers were expected to follow the course that was marked by signs having the message *TEST ROAD*. Drivers were not specifically informed that the observer would guide them if they missed a turn, nor was a missed turn called to their attention;

Figure 5.—Dummy sign, black legend on reflectorized white background.

Figure 6.—Decoy sign, similar in design and color to test signs, no test message.

Figure 7.—Auxiliary sign, for ramps and other changes in route.

the observer merely began giving corrective directions. Where there was a choice of roadways, drivers were further instructed to stay on the main road except where directed otherwise by signs or by the observer. By giving the drivers a plausible reason for following the signs, it was hoped that the explanation would not only somewhat satisfy their understandable curiosity, but that it would also minimize emphasis on the fact that the signs themselves were the object of the study. To further obtain consistency, operators were instructed to keep their headlights on low beam during the study. Subjects drove the route only once during the test, each being dispatched at 5-minute intervals to prevent overtaking.

To reduce exposure to the public, the signs were put in place at approximately 6 p.m. and taken down after the last subject had traversed the route.

At the beginning of the test, the observer directed the subject to drive to the exit of the staging area parking lot where one of the small auxiliary signs read *TEST ROAD*. From this point on, the driver was on the test route. The observer recorded turns made correctly or missed. Whenever an error was made, the operator was directed back onto the route in such a way that he would not pass by any of the remaining test signs out of the proper sequence. After he returned to the staging area, a second questionnaire was completed that dealt with such items as the subject's familiarity with the area of the route and his problems in following the route.

Reflective Properties of the Test Sign Materials

The specific luminance curves shown in figure 3 were derived from outdoor measurements on 24-inch-square plaques identical to the materials used for the test sign. Similar measurements made on a sample basis of the actual test signs were compared with those on the plaques and were essentially the same. The curves plotted were derived from the measurements on the plaques. Because of the limited number of measurements on these materials and the accuracy of the available instrumentation, the curves shown are representative of the relative specific luminances of the materials rather than the absolute specific luminances.

The luminance values for material I, the nonreflectorized background material, appears somewhat high, probably because the material was a smooth sheeting and produced some specular reflection. However, when used on the test signs, the same characteristics were exhibited.

Data on figure 8 show how the brightness of the sign materials varied with distance from the sign for each turn indicated. These data represent actual field measurements of the 24-inch-square plaques mounted on the sign supports and illuminated by the low beams of a single vehicle. While the vehicle was stationary at a measured distance from the sign, brightness measurements were made by a Pritchard photometer mounted at the driver's head position. The plotted numbers represent the turn at which the brightnesses were obtained for material III, the standard sheeting sign background, at the indicated distances from the signs. The solid curve has been drawn through the median brightness for all turns at each distance for this material (the median value for the 700-foot distance is based only on those signs that were not obscured at that distance). The median curves for material II, the intermediate sheeting, and the white legend material, respectively, are shown by broken lines below and above the curve for the standard sheeting.

The large differences in brightness for a given material at a specified distance were caused by the variations in horizontal and vertical curvature on the approaches to the signs at the individual sign locations. This variation in curvature caused differences in incidence angles and in the amount of headlamp illumination incident on the signs. To a lesser extent, variations in the approaches also caused differences in divergence angles. For each sign location and distance, the incident illumination and incidence and divergence angles were the same for each material, and thus the brightnesses were in the same ratio as the specific luminances. Although there is considerable overlap in the brightness for the three materials at different locations, the brightnesses of individual materials at the same location were in the same ratio. The median curves are therefore parallel, and the individual measured points for the other two materials, if shown, would be as far from the corresponding turn points indicated for the standard sheeting as the distances between the median curves for the other two materials. The brightness values of the three materials at each location were obtained under low-beam illumination by a single vehicle and are therefore minimum values. The brightnesses of the signs during the tests could have been higher as a result of illumination contributed by other vehicles on the highway.

Volunteers

The number of test subjects was limited by the need to run the study in multiples of 3 nights so that all three background treatments could be used an equal number of times at each turn. It was not feasible to extend the test to 6 or more nights because of increased field crew requirements and possible publicity of the test project. The 5-minute intervals between the start of each test run permitted

Table 3.—Number of errors, by turns, nights, and background

Turn number	All errors							All errors, less doubtfuls						
	1st night	2d night	3d night	Background I	Background II	Background III	Errors by turns	1st night	2d night	3d night	Background I	Background II	Background III	Errors by turns
1	16	19	19	16	19	19	54	13	15	18	13	15	18	46
2	11	7	13	11	13	7	31	10	5	11	10	11	5	26
3	16	8	13	8	13	16	37	14	3	11	3	11	14	28
4	3	4	4	4	4	3	11	2	4	4	4	4	2	10
5	4	8	4	4	8	4	16	4	6	4	4	6	4	14
6	1	2	3	2	1	3	6	1	2	2	2	1	2	5
7	1	2	2	2	2	1	5	1	2	2	2	2	1	5
8	1	1	2	1	2	1	4	1	1	2	1	2	1	4
9	5	1	0	5	1	0	6	4	1	0	4	1	0	5
10	2	2	2	2	2	2	6	2	2	2	2	2	2	6
11	1	4	5	5	1	4	10	1	3	4	4	1	3	8
12	10	7	9	7	10	9	26	5	6	4	6	5	4	15
13	1	1	0	0	1	1	2	1	1	0	0	1	1	2
14	1	0	0	0	0	1	1	1	0	0	0	0	1	1
15	1	0	0	1	0	0	1	1	0	0	1	0	0	1
16	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17	1	4	2	2	1	4	7	1	2	1	1	1	2	4
18	0	1	1	1	1	0	2	0	1	1	1	1	0	2
SUMMARY:														
Turns 1–18	75	71	79	71	79	75	225	62	54	66	58	64	60	182
Turns 1–3	43	34	45	35	45	42	122	37	23	40	26	37	37	100
Turns 4–18	32	37	34	36	34	33	103	25	31	26	32	27	23	82

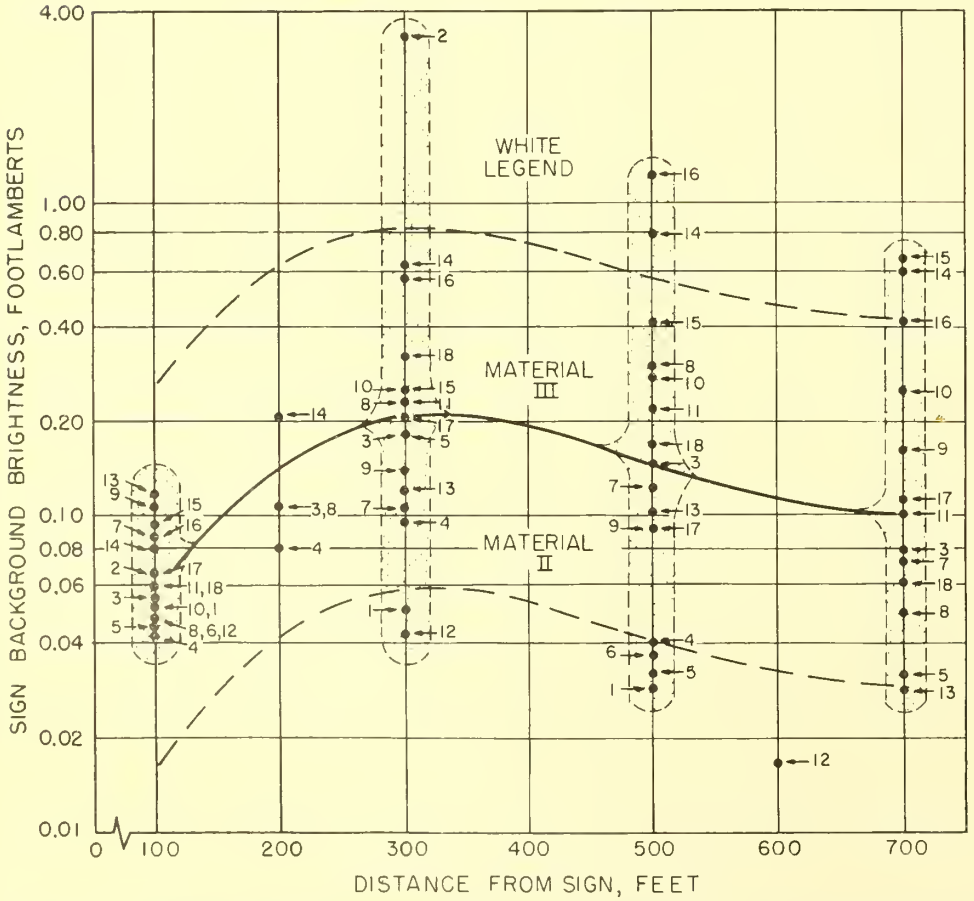

Figure 8.—Brightness of sign materials in place, by distance from signs for two-lamp, low-beam illumination. All plotted points are for material III, shown here as solid curve. Numbers next to points refer to turns where measurements were made. Dashed curves represent median brightnesses for material II and white legend.

about 60 subjects to be scheduled each night. Test runs began after 6:30 p.m., when the evening traffic peak had ended, and terminated shortly after midnight, which was as late as volunteers would remain on week nights.

Actually, more than 60 volunteers were tested on each of the 3 nights. However, because the last test sign was inadvertently removed on the first night while some test subjects were still on the route, all 60 test subjects were not completely exposed to the balanced experimental test design. In addition, a few *erratic* drivers missed almost all the turns and were hopelessly lost, and their data were not used. Complete data were collected for only 50 test subjects on the first night. To maintain a balanced experimental design, data for only the first 50 subjects on each of the other 2 nights were used.

Confounding factors

Several factors that may have confounded the results became apparent during the field operations or on inspection of the data sheets. Toward the middle of the runs in the second night and, to a lesser extent, later in the evening of the third night, temperature and humidity changes caused moisture condensation on many of the signs. This caused a lowering of the brightnesses of the reflective materials.

It became apparent, also, that some of the test subjects had not fully grasped the intent of the written instruction, which had emphasized the *TEST ROAD* message rather than the type of sign. Because the first sign having this message was the small auxiliary sign encountered at the exit from the staging

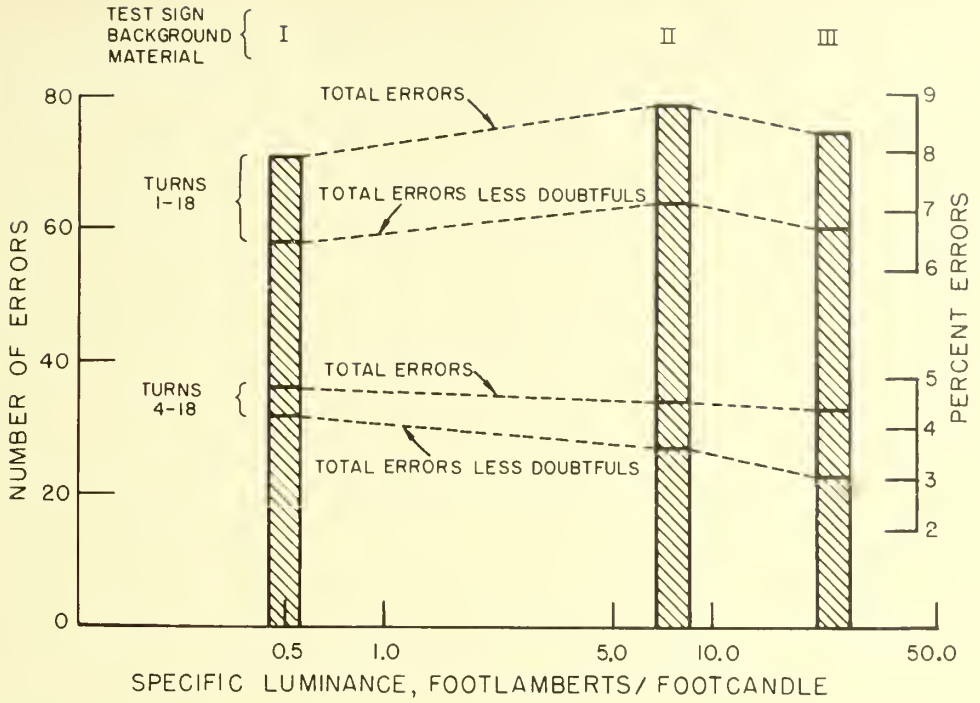

Figure 9.—Frequency and percentage of errors, by specific luminance of test sign background materials for different items of comparison. (Percent errors is based on 900 possible correct turns, for each material, for turns 1–18; and on 750 possible correct turns for each material, for turns 4–18.)

area, many subjects apparently formed a psychological set and, for the first few turns, looked only for the small white signs and ignored all others. This was substantiated by the fact that more than half of the total errors were made at the first three turns. Furthermore, an analysis of the data for consecutive turns missed showed that several subjects missed the first two or three turns before they realized that the test message appeared on other types of signs than the small auxiliary signs.

Finally, it became apparent that some of the exit gores and throats were poorly defined. Because the study was conducted in the middle of winter, the pavement markings were somewhat obliterated and the melting of previous snowfalls had left debris and dust at the curbs and road edges. (At the time of the tests, however, no snow was on the ground and the pavements were dry.) The observers recorded on the data sheets that the subject often called out when he saw the sign or reduced speed and operated his turn signal but failed to make the turn. From these comments, it was concluded that many of the errors were caused by the subject missing the exit rather than the sign. On the basis of the observer's comments, an attempt was made to cull out those errors that should not have been attributed to the signs. In the statistical analyses, these errors were termed *doubtfuls*.

Two analyses of errors were made by nights and by backgrounds for several categories: (1) data for turns 1 through 18, and (2) data for turns 4 through 18. Total errors and total errors from which doubtfuls were excluded were analyzed in each.

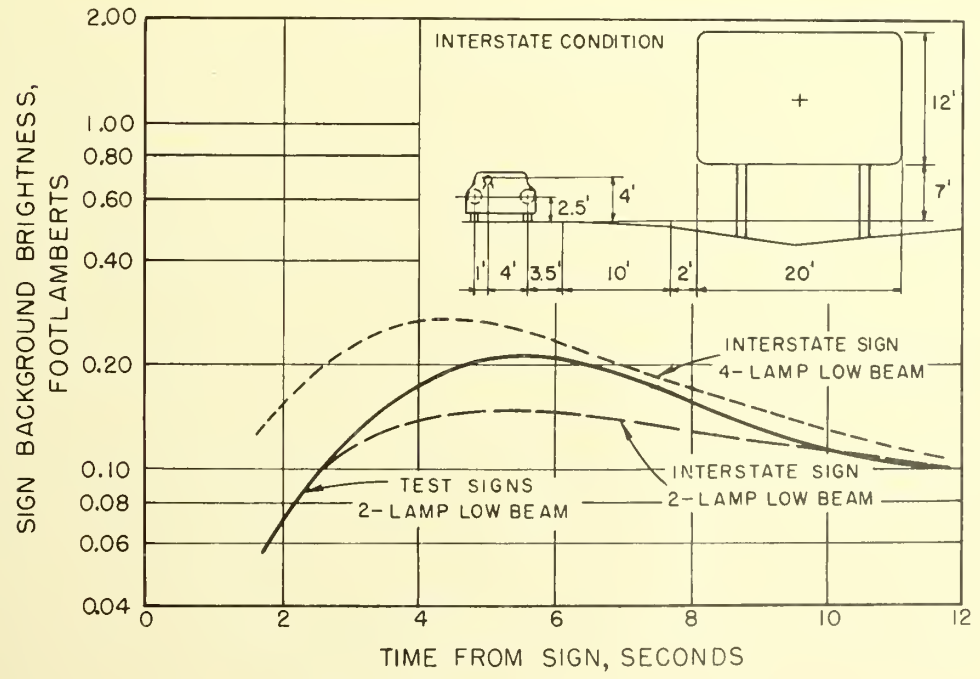

Figure 10.—Comparison of brightness of standard sheeting background material measured for test signs and calculated for typical Interstate highway signs. Assumed Interstate highway geometry used in calculations shown in inset.

Table 4.—Summary of chi-square tests

Item of comparison	X^2	Probability [1]
By nights:		
All errors—		
Turns 1–18	0.46	0.79
Turns 4–18	0.37	0.83
Doubtful errors omitted—		
Turns 1–18	1.34	0.51
Turns 4–18	0.78	0.69
By background material:		
All errors—		
Turns 1–18	0.46	0.79
Turns 4–18	0.14	0.93
Doubtfull errors omitted—		
Turns 1–18	0.31	0.86
Turns 4–18	1.54	0.47

[1] Probability shown is that of obtaining by chance alone a chi-square value as large as or larger than that observed; for example, a chi-square value as large as or larger than 0.46 would be expected to occur almost 8 times out of 10 merely from chance.

Data Analysis

The number of errors, by nights and by backgrounds, for each turn on the route and for selected groups of turns are shown in table 3. The results of chi-square tests[3] performed on these frequencies of errors are shown in table 4. The probability shown is that of obtaining by chance alone a chi-square result as large as or larger than that observed; for example, a chi-square statistic as large as or larger than 0.46 would be expected to occur almost 8 times out of 10 merely from chance.

The chi-square tests produced no evidence of any significant differences between the numbers of errors for any of the items of comparison, either by nights or by background treatments, as summarized in table 4. Chi-square tests on the relative frequencies of errors, by turns, were generally significant at the 1-percent level or less, indicating that there were significant differences in frequencies

[3] *Introduction to Statistical Analysis*, Wilfrid J. Dixon and Frank J. Massey, Jr., McGraw-Hill Book Co., Inc., New York, 1957, ch. 13.

of errors among the turns. These differences had been anticipated and the experiment designed accordingly.

The number of errors for the different items of comparison, as shown in figure 9, has been plotted by relative specific luminance of the background materials at a ½-degree divergence angle. The scales on the right side of the figure are the percentages of errors based on the number of sign encounters for each of the comparisons. Consideration of the errors for all turns shows the occurrence of errors to have been approximately 8½ percent of total possible errors. Omission of the doubtful errors brings the percentage down to 7. When the data for the first three turns are eliminated, total errors drop to approximately 4½ percent; and when the doubtfuls are omitted, total errors drop to 3¾ percent. Differences in the percentages of errors among the three background materials do not exceed 1 percentage point, and they are not statistically significant.

Discussion

This study was designed to test one aspect of the relative effectiveness of different degrees of sign background reflectorization: the ability of drivers to react correctly to information displayed on signs. Therefore, the analysis is directed primarily to a comparison of the relative frequency of errors made for signs in relation to different background materials. Differences that occurred in brightness for each of the sign locations were not part of the experimental design of the study. But, the same material will vary in brightness because of differences in highway alinement of the approach to the sign. Alinement affects the incidence angles and the intensity of headlight illumination on the signs. In fact, because of differences between locations, moderately reflective material II was brighter at some locations than material III (the standard sheeting) was at other locations.

Although the experimental design ensured that turns were balanced by background material, there was no balance between turns and brightnesses of the backgrounds. This was further confounded by the reduction in the brightness of some signs because of moisture condensation. However, because relative frequencies of errors by nights were not statistically significant, it is concluded that reductions in sign brightness did not cause an increase in errors. A plot of errors by sign background brightness (measured at 300 feet from the signs, the distance of maximum median brightness) showed such scatter that no definite trend could be determined. A possible trend toward fewer errors at higher background brightness was indicated by the plot of errors, but the small number of locations at which high background brightness occurred and the lack of balance between turns and brightness made an analysis of errors by brightness almost meaningless.

The measured brightness of the sign materials on signs in place are shown in figure 8. Although analysis of the study data showed no evidence of any difference in relative frequency of errors between materials for the brightnesses covered, it is possible that fewer errors would have been made if the brightness level had been higher. Before considering this possibility, it might be well to compare the sign brightnesses recorded in the study with those that would exist in actual practice on an Interstate highway. To compare the relative brightness of the same material under test conditions and under Interstate conditions, the standard sheeting median brightness curve for the test signs from figure 8 and brightness curves of standard sheeting for Interstate signs, calculated for four-lamp and two-lamp low beams, have been plotted in figure 10. To allow for differences between speeds of 40 m.p.h. on the test route and 60 m.p.h. on Interstate highways, the abscissa is in units of time elapsed before the sign is reached. This illustrates that, for the same material, the brightnesses of the test signs and those of Interstate signs were comparable. Comparison of the curves for two-lamp illumination shows that the test signs were, on the average, slightly brighter than the Interstate signs. Therefore, sign backgrounds brighter than those tested would have had to have more brightness than ordinarily is available from standard sheeting on Interstate signs.

As previously stated, the highways on which the tests were conducted were designed to lower standards than those for current Interstate highways, and prevailing speeds were also lower. Compared to Interstate conditions, speeds on the test route were approximately two-thirds of those on Interstate highways; sign legends were about one-half the size; and the signs themselves were about one-fourth the size of Interstate signs. The size ratio between the test signs and Interstate signs was smaller than the ratio between the speeds on the respective facilities. Therefore, except for the closely spaced exits, study conditions may have constituted a more difficult test of signing than current Interstate conditions impose.

The methodology employed for the study discussed here may have applicability to other studies in related subject areas. A relatively large number of test subjects would be needed, however, because of the small proportion of typical errors observed. As in all studies involving test subjects as drivers, precise and concise instructions are essential to minimize confusion. Temperature, humidity, and precipitation are additional factors that must be considered, although they are usually outside the realm of control.

Findings

The findings obtained from this study are limited because of the relatively few errors made and the presence of confounding factors in the tests. Errors made by test subjects in following the route were not statistically significant by type of sign background. Therefore, the study findings do not substantiate the existence of any difference in the effectiveness of different degrees of sign background reflectorization. Because all data on missed turns are enumeration data and therefore insensitive to small differences, more data—more subjects or more turns—would be required to establish statistical significance.

The occurrence of errors, in absolute terms, was relatively small: errors made, from whatever source, averaged 8½ percent of total possible errors; and errors attributable to the signs and not to the conditions of the study amounted to less than 4 percent. If the probability of missing a single sign, regardless of the degree of background reflectorization, is 4 percent, the probability of missing two advance signs would be in the order of 0.16 percent.

Based on the observers' comments, one of the major problems in providing guidance to drivers is to enable them to relate the information on the sign and the placement of the sign to a decision to change direction. Another problem facing drivers is that of locating the geometric features to which the sign relates, particularly the exit gore.

ACKNOWLEDGMENTS

This study could not have been made without the assistance and cooperation of many individuals and organizations. Particular appreciation is extended to Richard N. Schwab of the Traffic Systems Division for his assistance during the organization and execution of the study and for his direction of the visual and photometric tests. Appreciation is also extended to Region 15 of the Bureau of Public Roads for the fabrication and erection of the sign supports and for the placement and removal of signs during the study; to the Arlington County Highway Division, the Virginia Department of Highways, and the National Park Service of the Department of the Interior for permission to erect signs on highways under their jurisdictions; to Government Services, Inc., for the use of the Columbia Island Marina Parking Lot and Boathouse for a staging area; to the District of Columbia Department of Highways and Traffic for the fabrication of the auxiliary signs used in the study; to the Junior Chambers of Commerce of the District of Columbia and Maryland suburbs, local chapters of the Lions Club, and the local chapter of the Sports Car Club of America for their cooperation in communicating to their members the need for test subjects; to the individuals who volunteered to be test subjects; to the group of men who, during their period of training as Junior Engineers in the Washington office of the Bureau of Public Roads, took part in the study; and to Sterling Seagrave, reporter for the Washington Post & Times Herald newspaper, for withholding his article on the study until after completion of the field test.

Shear Loads on Pavements

by EDWARD S. BARBER, Highway Research Engineer,[1]
Structures and Applied Mechanics Division

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

INFLUENCE charts are presented here for determining the horizontal normal stresses from a uniform shear load over any area. These data are presented as a supplement to *Shear Loads on Pavements*, by E. S. Barber, in PUBLIC ROADS, *A Journal of Highway Research*, vol. 32, No. 6, February 1963, pp. 141–144, in which information on the vertical normal and horizontal shear stresses from a shear load is given (p. 143 and fig. 9).

The charts are similar to those developed by Newmark [2] for a vertical applied load. To calculate horizontal normal stress in a direction parallel to the direction of the load, use figure 1 for Poisson's ratio of 0.5 and figure 2 for Poisson's ratio of 0. Values for other Poisson's ratios are determined by linear interpolation, which is theoretically exact for this load.

To determine the stress below point A at a depth of A–Z, plot the loaded area on the chart by using the distance A–Z=depth, as the plotting scale. Two scales A–Z are shown so that unequal shrinkage of paper will not affect calculations. Count the number of blocks covered by the loaded area. The transmitted stress is 0.001 × number of blocks × applied shear stress. The influence is symmetrical about the lower edge of the chart; any loaded area below the edge is folded about the lower edge onto the region above the edge. There is similar symmetry about the left edge, except that the influence to the left of the left edge is negative (tension). The small area in figure 2 between point A and the dotted curve has a very small negative influence.

Use figures 3 and 4 to determine the horizontal normal stress in a direction perpendicular to the direction of the applied load for Poisson's ratio equal, respectively, to 0.5 and 0. There is symmetry about the lower and left edges as in figures 1 and 2. In figure 4, the influence of the blocks covered with pattern is tension.

[1] Mr. Barber is also Associate Professor of Civil Engineering at the University of Maryland, College Park, Md.

[2] Influence Charts for Computation of Stresses in Elastic Foundations, by N. M. Newmark, University of Illinois Eng. Expt. Sta. Bul. No. 338, 1942.

Aerial Color Photography and Its Use in Materials Surveys

(continued from page 171)

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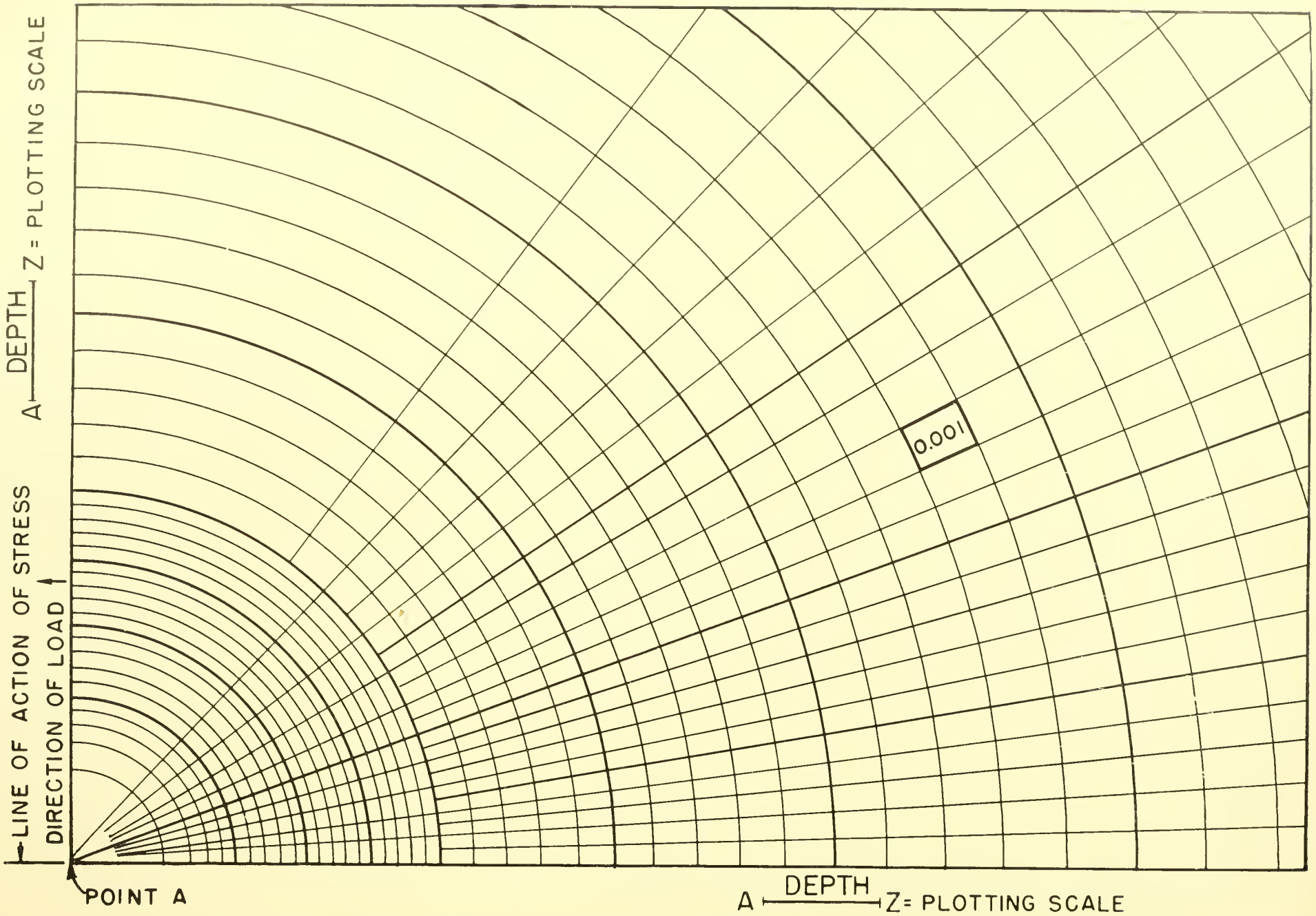


Figure 1.—Influence Chart for horizontal normal stress from applied shear load, stress parallel to load—Poisson's ratio, 0.5.

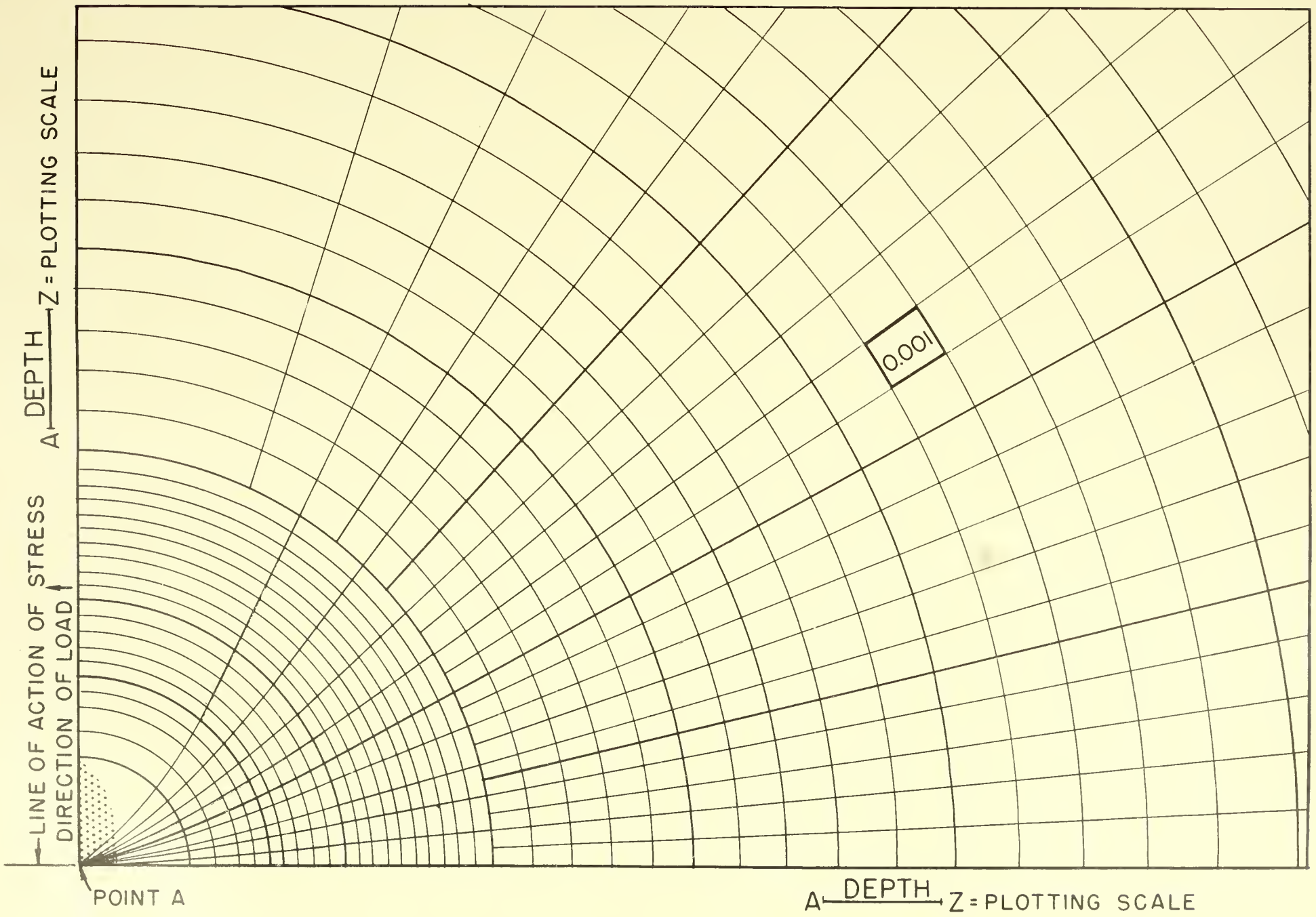


Figure 2.—Influence Chart for horizontal normal stress from applied shear load, stress parallel to load—Poisson's ratio, 0.

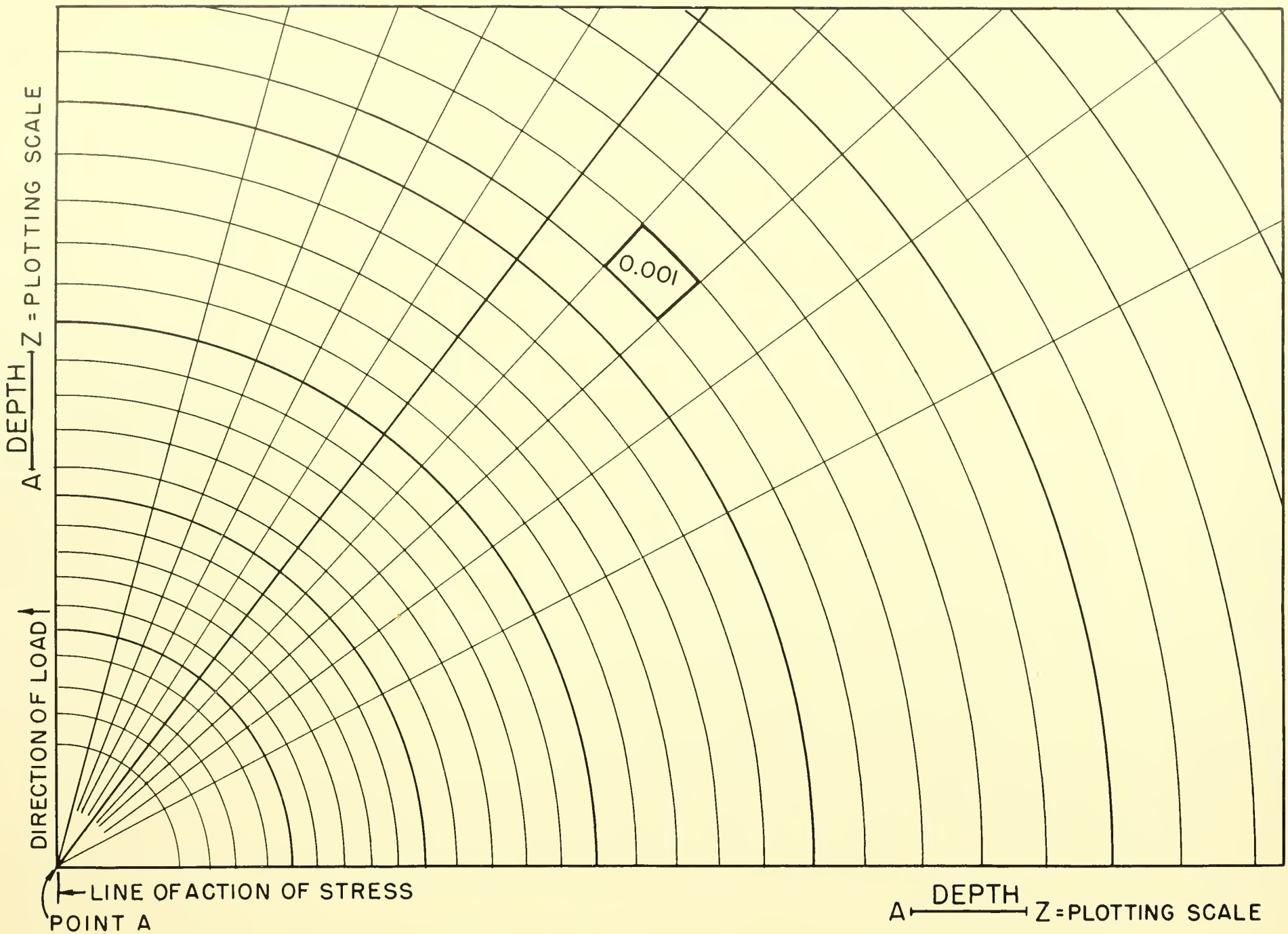


Figure 3.—Influence Chart for horizontal normal stress from applied shear load, stress perpendicular to load—Poisson's ratio, 0.5.

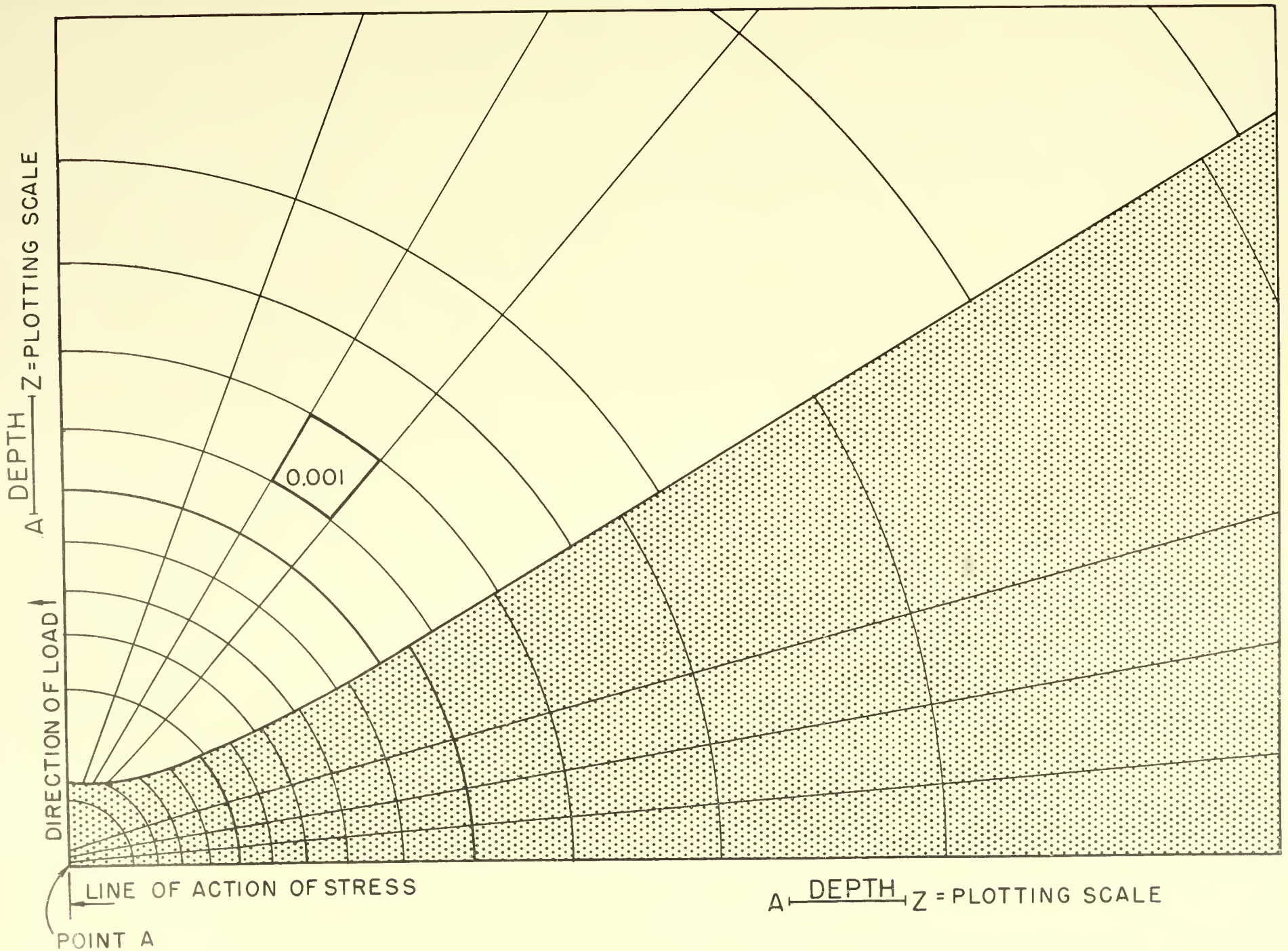
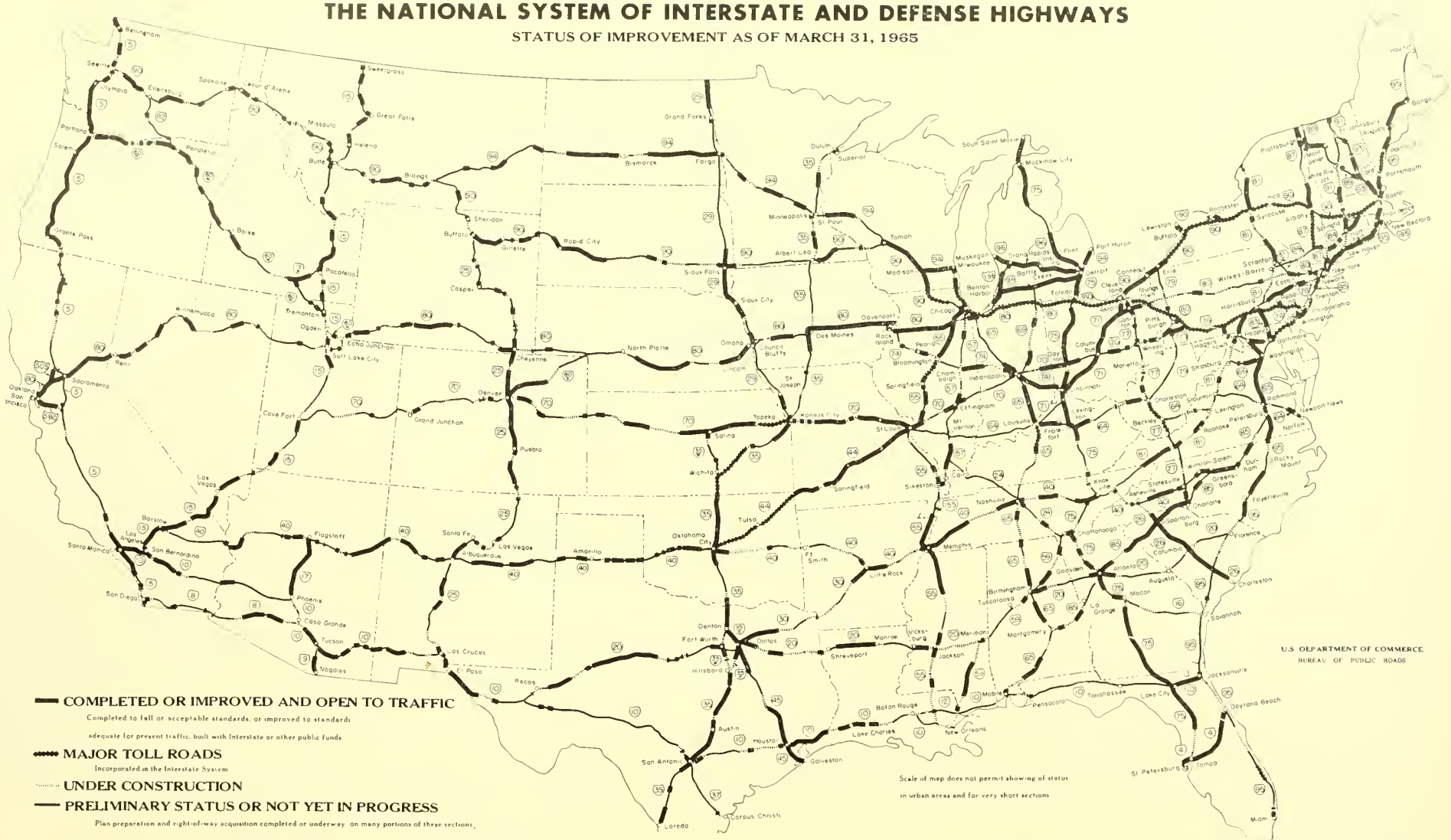


Figure 1.—Influence Chart for horizontal normal stress from applied shear load, stress perpendicular to load—Poisson's ratio, 0.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF MARCH 31, 1965


COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards, or improved to standards adequate for present traffic, built with Interstate or other public funds
MAJOR TOLL ROADS
Incorporated in the Interstate System
UNDER CONSTRUCTION
PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right-of-way acquisition completed or underway on many portions of these sections.
Scale of map does not permit showing of status in urban areas and for very short sections
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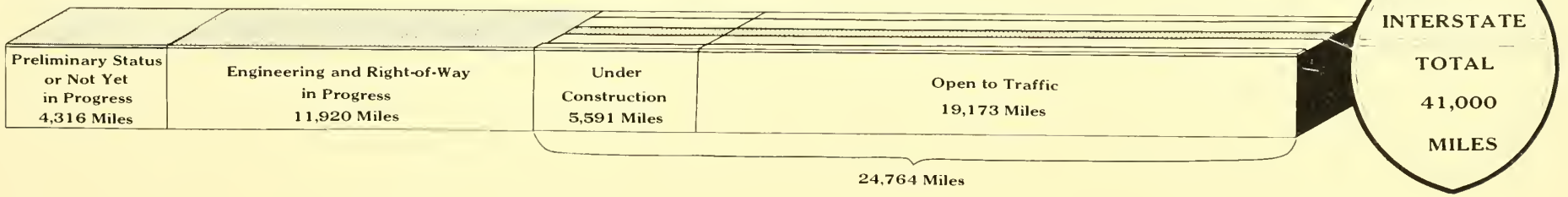
Preliminary Status or Not Yet in Progress 4,316 Miles
Engineering and Right-of-Way in Progress 11,920 Miles
Under Construction 5,591 Miles
Open to Traffic 19,173 Miles
24,764 Miles
INTERSTATE TOTAL 41,000 MILES

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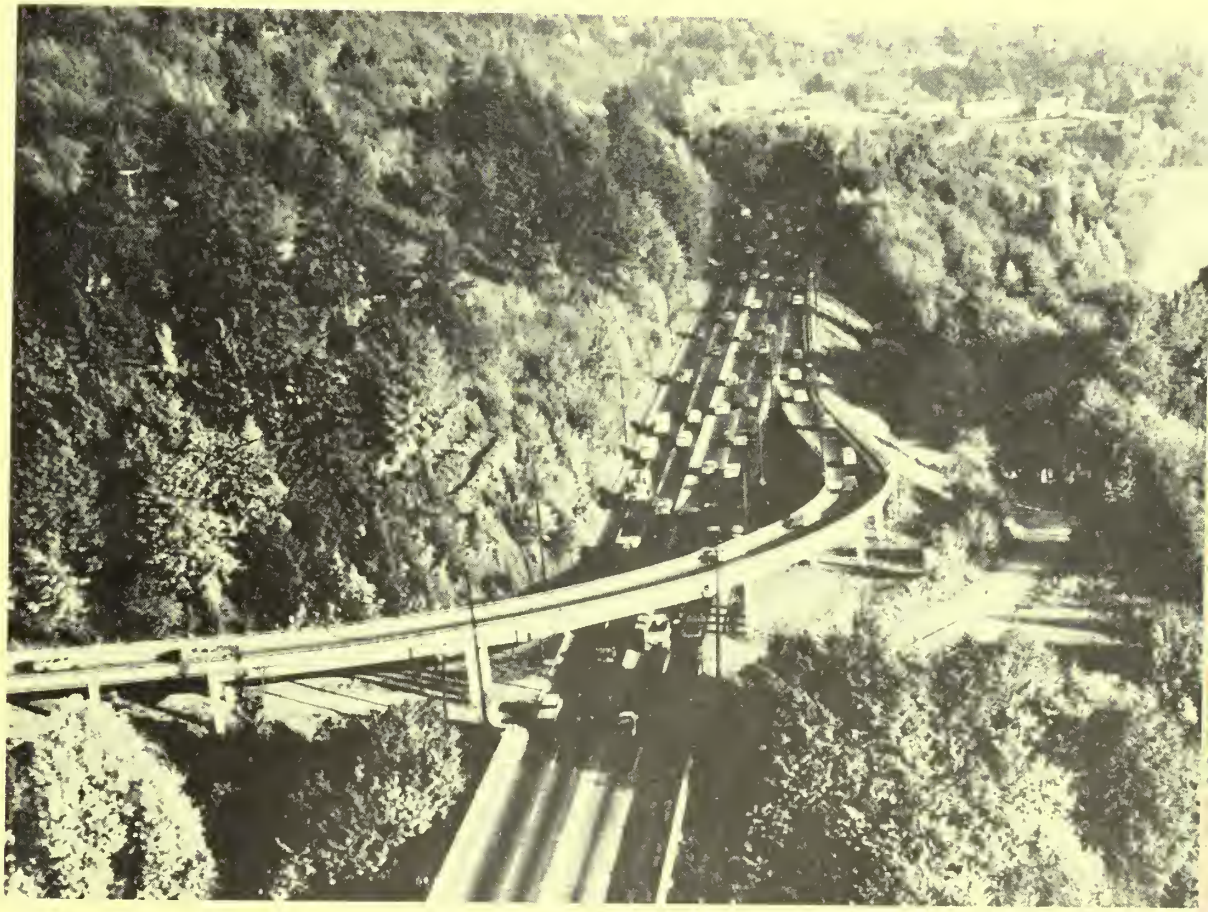

Overhead ramp from Slavin Road to Barbur Boulevard, Portland, Oreg.

A dramatic reduction in accidents was brought about by installation of this overhead ramp. Prior to its construction, access was at grade with traffic light control. Accidents averaged 120 a year, with 60 injuries. The number of accidents has now dropped 96 percent to only 4 a year, with 2 injuries, while average daily vehicle counts have increased from 11,000 to 13,000.

AUG 25 1965

CLEMSON, SOUTH CAROLINA

IN THIS ISSUE

Highway Income, Expenditures, and Earnings in 16 Standard Metropolitan Statistical Areas. *by S. F. Bielak and J. F. McCarthy* 185

Motor Vehicle Size and Weight Limits (AASHO table on pages 192–193) 200

New Publications 200

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Contents of this publication may be reprinted. Mention of source is requested.

Highway Income, Expenditures, and Highway-User Earnings in 46 Standard Metropolitan Statistical Areas

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

Reported by [1] STANLEY F. BIELAK, Chief, Financial Branch, and JAMES F. McCARTHY, Economist, National Highway Planning Division

Information is presented in this article on road-user tax earnings, highway income, and highway expenditures in 46 Standard Metropolitan Statistical Areas, in 1960. This information was collected to obtain a basis for determining whether an equitable contribution for highway construction was being made by urban and rural highway users. Road-user taxes, fees, and tolls are expressed as earnings of road-user taxes, and these revenues are discussed in relation to highway income from all sources and to expenditures for roads and streets, by population groups.

The road-user revenues from motor-vehicle travel in the 46 SMSA's amounted to $1.65 billion, but highway expenditures were only $1.49 billion. Total highway revenue allocated to the SMSA's was $1.42 billion. Therefore, earnings from motor-vehicle use in these areas exceeded the amount assigned for highways, even when income from borrowings was included. Earnings of road-user taxes also exceeded total highway expenditures, which included debt retirement.

The most favorable ratio of earnings to expenditures, by SMSA population groups, was in the population group 500,000 to 1,000,000, 1.16:1; the population group of 1,000,000 and more had an earnings-expenditure ratio of 1.13:1; the lowest was in the population group of less than 250,000, 0.91:1; and for all SMSA's the ratio of earnings to expenditures was 1.11:1.

On the basis of the data for the 46 SMSA's, the balance of user earnings from travel in metropolitan areas is favorable when compared to the high per-mile costs of construction of urban highway facilities during an accelerated period of construction activity.

Introduction

GROWTH in population, motor-vehicle registrations, and travel in urban areas has been occurring at such a rapid rate that increased allocations of funds for highways have become necessary. The high cost per mile of urban highway facilities, because of the uniform rate of Federal and State user charges on rural and urban residents, prompted an investigation of highway finances in Standard Metropolitan Statistical Areas (SMSA's).[2] By using the local and State highway finance data that have become available recently, information is presented in this article to relate highway income to highway expenditures and earnings of highway-user taxes in 46 Standard Metropolitan Statistical Areas. Although this article provides only a brief glance at the total highway financial picture, it focuses on an area of highway finance that has not been extensively explored. No attempt has been made to include or evaluate social costs related to the costs of urban highway systems.

The authors have not presented any direct conclusions, but a statistical summary is included at the end of the article.

Procedure

To determine how highway-user earnings, highway income, and expenditures are related in urban areas, the SMSA was adopted as the unit of measure. Of the 212 SMSA's defined in the 1960 Census, exclusive of Puerto Rico, 46 were used as the sample for the study reported here. Information collected for the 46 SMSA's and the same information for the 212 SMSA's is given in table 1. The sample used represents nearly 22 percent of the SMSA's and a little more than 31 percent of the total SMSA population. The sample, as shown by the table, is somewhat weighted in favor of the more populous areas—37 percent of the population of SMSA's having populations of more than 1 million, and only 20 percent of the SMSA's having populations of less than 250,000. This disparity has been somewhat minimized in the material presented here as the SMSA's have been grouped according to population and the discussion is related to each of these population groupings individually.

The terms *earnings* or *road-user tax earnings* as used in this article refer to taxes levied on the use or ownership of motor vehicles. In this context, a fixed fee or annual charge, such as a motor-vehicle registration fee, operator's license, transfer fee, and other fees are credited to the SMSA where the vehicle is domiciled. A motor-fuel tax, taxes on tires and other components that are consumed by travel, and tolls are credited to the SMSA where the travel occurs.

All of the States were requested to select and to report for one SMSA the total travel in 1960, subdivided where possible into travel by: (1) automobiles and (2) trucks and buses. They were also asked to give an estimated motor-fuel consumption rate for each of the two classes of motor vehicles. To obtain adequate travel data, the States were asked to report on an area in which a transportation study had been recently completed or was sufficiently advanced to be of aid in preparing the travel estimates.

The 46 SMSA's included in this analysis represent 1 in each of 44 States, and 2 in Indiana, as illustrated in figure 1. New Hampshire did not provide data, and in 1960 there were no SMSA's in Alaska, Idaho, Vermont, and Wyoming. Although the selection on this basis does not sample the geographic or population areas to the same degree, a more representative cross section of other

[1] Presented at the 44th annual meeting of the Highway Research Board, Washington, D.C., January 1965.

[2] A Standard Metropolitan Statistical Area, established by the Bureau of the Budget for convenience of reporting, consists of the counties, or towns in New England, that contain the entire urbanized portion of a metropolitan area. The SMSA includes, of necessity, the rural portion, if any, of its constitutent counties.

Figure 1.—Geographic distribution of 46 SMSA's.

characteristics is obtained, some of which are listed in table 2. By sampling each State, it was possible to obtain data on: (1) the diverse State motor-fuel and motor-vehicle tax rates; (2) a variety of construction programs, particularly on The National System of Interstate and Defense Highways where in a given year construction activity in the urban areas of some States may greatly exceed that in others; (3) a sample of areas in which the central cities originated and developed at different times, such as the older eastern cities and the newer and rapidly growing cities in western areas; and (4) those cities that have urban transportation systems developed around rails and highways, as well as those where transportation is mainly highway oriented.

Data Used

Information concerning the 46 SMSA's selected for a sample is given in table 2. These details are presented to illustrate the differences in makeup of the individual SMSA's. The data for population, land area, and motor-vehicle registrations are a matter of record, except in a few areas where motor-vehicle registrations were estimated. The data on vehicle-miles of travel are perhaps the most uncertain link, but they seem to be acceptably consistent. The extreme rate of 20.0 miles of travel per person per day in the Atlantic City, N.J. Area, is attributable to the very heavy seasonal use by nonresidents. This figure and the corresponding figure of 20,372 miles of annual travel in the Atlantic City Area, per vehicle registered there, point up the fact that the denominators of these ratios are somewhat defective because the travel in an SMSA includes that of visitors as well as residents. However, the low rate of 7.8 miles per person per day in the Philadelphia Area is reasonably comparable with 11.3 miles in the Los Angeles Area and reflects the populations' use of transit facilities and a

Table 1.—Data on the 46 selected SMSA's and all SMSA's in the United States [1]

Census region and population group	SMSA's and population						Land area			Population per sq. mi.	
	All SMSA's		Study sample		Sample, percent of total		All SMSA's	Study sample	Sample, percent of total	All SMSA's	Study sample
	Number	*Population (thousands)*	*Number*	*Population (thousands)*	*Number (percent)*	*Population (percent)*	*Sq. mi.*	*Sq. mi.*	*Percent*	*Number*	*Number*
All SMSA's	212	112,885	46	35,246	21.7	31.2	310,233	75,855	24.5	364	465
Census regions:											
Northeast	47	35,347	7	6,961	14.9	19.7	35,650	6,746	18.9	991	1,032
North Central	59	30,960	13	10,443	22.0	33.7	87,834	16,678	19.0	352	626
South [2]	77	26,447	16	7,676	20.8	29.0	59,328	15,351	25.9	446	500
West	29	20,131	10	10,166	34.5	50.5	127,421	37,080	29.1	158	274
Population groups:											
More than 1,000,000	24	61,582	7	23,065	29.2	37.5	54,285	19,321	35.6	1,134	1,194
500,000 to 1,000,000	29	19,215	7	5,096	24.1	26.5	70,767	16,896	23.9	272	302
250,000 to 500,000 [2]	48	15,829	11	3,901	22.9	24.6	78,460	11,219	14.3	202	348
Less than 250,000	111	16,259	21	3,184	18.9	19.6	106,721	28,419	26.6	152	112

[1] Excludes Puerto Rico.

[2] Population and area of Osage County of the Tulsa, Okla., SMSA are not included in the sample but are in the totals of all SMSA's in the United States.

much later development of freeways in the Philadelphia Area than in the Los Angeles Area.

Travel data

Each State was requested to report the total motor-vehicle travel on all roads and streets of the selected SMSA for the calendar year 1960. The State was also asked to classify the travel by that: (1) on the rural roads, (2) on the urban highways and streets, (3) of automobiles, and (4) of trucks and buses combined. The responses by the States were different in degrees of detail that ranged from travel classified by vehicle types and by road systems to only the total vehicle-miles of travel and percentages that indicated distribution of the total travel between automobiles and trucks and buses.

The method of estimating and classifying the travel in the SMSA's also differed. For States in which some form of area transportation studies were available, the information was applied to the 1960 data by travel trends; in others, estimates were prepared from available information on mileage of local streets and arterials and the corresponding current travel volumes on them. Generally, too, where data from area transportation studies were utilized, it was necessary to supplement them with travel in the area beyond that study's external cordon to the county boundaries forming the SMSA. However, it is believed that sufficient accuracy was obtained because the routes that carry the bulk of the travel are the State highways and primary local roads for which data were available from current traffic-counting programs, and these outlying areas were predominantly rural.

Motor-vehicle registrations

Registrations of motor vehicles by counties are compiled by the States and are currently available for approximately 41 States. In the other States the SMSA registrations were estimated by using collateral data of the Bureau of the Census (*1*),[3] and the annual and special reports of State motor-vehicle registrations (*2*).

Road and street income

The income for road and street purposes of an SMSA comes from several sources. The accounting of the income for each SMSA is obtained by the State highway departments from State and local records and summarized in reports transmitted annually to the Bureau of Public Roads. Income and expenditure data of local governments are summarized in this article from forms PR–532, State Highway Expenditures, and PR-535, Local Road and Street Finance Report. For each SMSA the PR-535 report includes the annual receipts, disbursements, obligations issued, application of proceeds, and a statement of interest and bond redemptions. State income from road users equivalent to State expenditures for highways given in form PR-532-B, State Highway Expenditures Within Standard Metropolitan Statistical Areas, is assigned from State and Federal user revenues, as explained subsequently.

In this article income for highways is classified according to (1) the imposts on highway users collected at the different government levels—Federal, State, and local—and tolls on State and local facilities; and (2) other revenue income of an SMSA, consisting of property taxes and assessments, general fund appropriations—State and local—and miscellaneous local income from a variety of sources such as subdivider payments for road improvements, fines for parking meter violations, rentals, excavation permits, utility taxes, adjustments and repairs, and, in some areas, a miscellany

[3] References indicated by italic numbers in parentheses are listed on p. 199.

Table 2.—Information on population, land area, registered vehicles, and travel in 46 SMSA's, 1960

SMSA's, by population group	Population	Land area	Persons per square mile	Registered vehicles	Travel	Persons per vehicle	Travel	
							Person per day	Annual, in SMSA per registered vehicle therein
LESS THAN 250,000								
	Number	*Sq. mi.*	*Number*	*Number*	*Veh.-miles (thousands)*	*Number*	*Miles*	*Miles*
Atlantic City, N.J.	160,880	575	280	57,678	1,175,000	2.8	20.0	20,372
Bay City, Mich.	107,042	446	240	44,280	425,000	2.4	10.9	9,598
Cedar Rapids, Iowa	136,899	713	192	63,557	501,680	2.2	10.0	7,893
Charleston, S.C.	216,382	945	229	67,766	715,000	3.2	9.1	10,551
Eugene, Oreg.	162,890	4,560	36	85,003	643,400	1.9	10.8	7,569
Fargo, N. Dak.	106,027	2,799	38	51,492	451,962	2.1	11.7	8,777
Fitchburg-Leominster, Mass.	82,486	99	833	28,479	426,000	2.9	14.1	14,958
Fort Wayne, Ind.	232,196	670	347	99,016	581,960	2.4	6.9	5,877
Great Falls, Mont.	73,418	2,659	28	35,904	273,057	2.0	10.2	7,605
Jackson, Miss.	187,045	877	213	70,890	589,712	2.6	8.6	8,319
Las Vegas, Nev.	127,016	7,927	16	75,750	358,823	1.7	7.7	4,737
Lewiston-Auburn, Maine	70,295	120	586	24,167	174,125	2.9	6.8	7,205
Lexington, Ky.	131,906	280	471	53,644	432,700	2.5	9.0	8,066
Little Rock-No. Little Rock, Ark.	242,980	767	317	103,603	795,700	2.3	9.0	7,680
Lynchburg, Va.	110,701	1,014	109	37,168	423,912	3.0	10.5	11,405
Macon, Ga.	180,403	630	286	66,077	451,870	2.7	6.9	6,839
Madison, Wis.	222,095	1,197	186	87,628	911,610	2.5	11.2	10,403
Sioux Falls, S. Dak.	86,575	815	106	40,403	340,451	2.1	10.8	8,426
South Bend, Ind.	238,614	467	511	98,138	570,090	2.4	6.5	5,809
Springfield, Mo.	126,276	677	187	56,713	659,096	2.2	14.3	11,622
Waterbury, Conn.	181,638	182	998	76,696	523,283	2.4	7.9	6,823
Total	3,183,764	28,419	112	1,324,052	11,424,431	2.4	9.8	8,628
250,000 TO 500,000								
Albuquerque, N. Mex.	262,199	1,163	225	109,249	827,424	2.4	8.6	7,574
Charleston, W. Va.	252,925	908	279	86,166	814,431	2.9	8.8	9,452
Charlotte, N.C.	272,111	542	502	120,599	675,129	2.3	6.8	5,598
Jacksonville, Fla.	455,411	777	586	187,524	1,807,115	2.4	10.9	9,637
Nashville, Tenn.	399,743	532	751	147,128	1,208,996	2.7	8.3	8,217
Omaha, Nebr.	457,873	1,533	299	189,698	1,842,338	2.4	11.0	9,712
Salt Lake City, Utah	383,035	764	501	174,021	1,155,000	2.2	8.3	6,637
Tacoma, Wash.	321,590	1,676	192	134,292	1,281,000	2.4	10.9	9,539
Tulsa, Okla.[1]	386,533	1,538	251	187,975	1,436,382	2.1	10.2	7,641
Wichita, Kans.	343,231	999	344	161,042	1,381,796	2.1	11.0	8,580
Wilmington, Del.	366,157	787	465	139,170	1,586,247	2.6	11.9	11,398
Total	3,900,808	11,219	348	1,636,864	14,015,858	2.4	9.8	8,563
500,000 TO 1,000,000								
Birmingham, Ala.	634,864	1,118	568	234,198	2,052,312	2.7	8.9	8,763
Columbus, Ohio	682,962	537	1,272	282,428	2,696,374	2.4	10.8	9,547
Denver, Colo.	929,383	3,665	254	465,125	3,500,000	2.0	10.3	7,525
Honolulu, Hawaii	500,409	598	837	175,676	1,123,090	2.8	6.1	6,393
New Orleans, La.	868,480	1,118	777	280,907	1,940,483	3.1	6.1	6,908
Phoenix, Ariz.	663,510	9,226	72	336,465	3,083,304	2.0	12.7	9,164
Providence, R.I.	816,148	634	1,287	318,539	3,401,100	2.6	11.4	10,677
Total	5,095,756	16,896	302	2,093,338	17,796,663	2.4	9.6	8,502
MORE THAN 1,000,000								
Baltimore, Md.	1,727,023	1,807	956	572,478	5,965,707	3.0	9.5	10,421
Buffalo, N.Y.	1,306,957	1,587	824	448,307	3,417,680	2.9	7.2	7,624
Chicago, Ill.	6,220,913	3,714	1,675	2,083,209	19,210,133	3.0	8.5	9,221
Houston, Texas	1,243,158	1,711	727	572,343	4,265,000	2.2	9.5	7,452
Los Angeles, Calif.	6,742,696	4,842	1,393	3,415,201	27,808,000	2.0	11.3	8,142
Minneapolis-St. Paul, Minn.	1,482,030	2,111	702	642,617	5,500,000	2.3	10.2	8,559
Philadelphia, Pa.	4,342,897	3,549	1,224	1,536,952	12,313,914	2.8	7.8	8,012
Total	23,065,674	19,321	1,194	9,271,107	78,480,434	2.5	9.3	8,465
TOTAL, ALL SMSA's	35,246,002	75,855	465	14,325,361	121,717,386	2.5	9.5	8,497

[1] Does not include the population and area of Osage County of the Tulsa, Okla., SMSA.

Table 3.—Index of automobile motor-fuel consumption rates for overall operation and operation predominantly in rural and urban areas

Source	Consumption rates, gallons per mile			Index of consumption rates		
	Average	Rural	Urban	Average	Rural	Urban
Illinois MVU [1]	0.0725	0.0671	0.0813	1.00	0.93	1.12
Lieder [2]	.0690	.0625	.0785	1.00	.91	1.14
7-State MVU	.0669	.0616	.0724	1.00	.92	1.08
This study	[3].0700	.0650	.0756	1.00	.93	1.08

[1] *Illinois Motor Vehicle Use Study*, Illinois Division of Highways, October 1961, p. 205.
[2] *Passenger Car Fuel Consumption Rates*, by Nathan Lieder, PUBLIC ROADS, Vol. 32, No. 5, December 1962, p. 119
[3] Average obtained from *The Supplementary Report of the Highway Cost Allocation Study* (3). The rate for consumption in urban areas was developed for this analysis.

that includes traffic fines and other fees not segregated by a specific source.

Expenditures on roads and streets

The expenditures on roads and streets used for each SMSA are as complete as was possible from the available data. The roads and streets in these SMSA's are under several jurisdictions—State, county, and municipal. Road and street construction and maintenance is accomplished by one, two, or jointly by all three levels of government. To the extent that capital outlays are identified by system, they are listed in this article by State and local systems—rural and municipal. Expenditures for maintenance, operation, and administration are lumped because they are less easily identified. In the latter classification, local expenditures are complete, but State outlays are not.

State funds may be expended through: (1) capital outlay, which includes Federal aid; (2) maintenance by the State on State highway extensions in municipalities, on local rural roads, or municipal streets; or (3) grants-in-aid payments to local rural or municipal units, which are reflected in construction, maintenance, and administration expenditures at the local level. Funds are also transferred between local rural and municipal units, in addition to direct construction in each other's jurisdiction.

Local rural (county) and municipal highway administration, traffic police, bond service, and other miscellaneous expenditures are believed to be adequately represented in the reported data from the local records. Data for State and local toll facilities are available from the annual reports to Public Roads. For each facility situated entirely within an SMSA, the income and expenditures were used as recorded in the annual reports. However, for those facilities extending beyond the boundary of an SMSA, principally toll roads, the expenditures for all purposes such as construction, maintenance, and administration were assigned in the same proportion that the earnings within the SMSA had to the earnings of the entire facility.

Highway-user earnings

User taxes consist of a variety of levies—on the owner or operator of a vehicle, on the vehicle itself, or on the use of a vehicle. Registration fees, vehicle excise and use taxes, transfer and title charges, certain truck and bus franchise or user permits, and driver licenses are paid periodically and are a requirement for owning and operating a vehicle on the highways. Taxes on: gasoline and special fuel, truck and bus mileage, and tires and tubes are paid intermittently according to the number of miles the vehicle is operated.

The Federal excise taxes deposited in the Federal Trust Fund and designated for highway purposes are the 1960 user taxes for which earnings were evaluated in the study reported here. These excise taxes included: gasoline and special fuels at 4 cents per gallon; tires at 8 cents per pound; innertubes at 9 cents per pound; tread rubber at 3 cents per pound; truck, bus, and trailer excises at 5 percent of manufacturers' wholesale prices; and the vehicle-use tax at $1.50 per 1,000 pounds. Not included with these earnings are other Federal automotive excise taxes that accrue to the general fund, such as the automobile vehicle excise tax, parts and accessories tax, lubricating oil tax, and one-half of the truck, bus, and trailer excise tax at 5 percent of manufacturers' wholesale price.

State user charges consist of gasoline and special fuel taxes; mileage, ton-mile, and franchise taxes; registration fees; operator and chauffeur licenses; and miscellaneous charges for titling or transfer of ownership of vehicles.

Local road-user charges are not levied in all States or in all local jurisdictions of a State. Where imposed, they may consist of motor-fuel taxes, bus and wheel taxes, and licenses for automobiles and trucks. Although traffic fines and allied fees are often not considered to be regularly imposed user levies, they have been included with user taxes when they are identified and used for highway purposes.

In this article earnings based on use were computed for all travel in an SMSA regardless of where the motor vehicles were domiciled. Registration and other periodic charges used were those paid only for the vehicles domiciled within the SMSA.

Table 4.—Motor-fuel consumption rates applied to SMSA travel to obtain data on earnings from motor-fuel taxes

Vehicle class	Motor-fuel consumption rates				
	U.S. average gasoline and diesel vehicles [1]		Urban areas		
			Average		Ratio (U.S. average=1.0) [2]
	Gal./mile	*Miles/gal.*	*Gal./mile*	*Miles/gal.*	
Automobile	0.070	14.29	0.076	13.16	1.08
Transit bus	.237	4.22	.249	4.01	1.05
Intercity bus	.167	5.99	.215	4.65	1.29
School and other bus	.129	7.75	.129	7.75	1.00
Trucks and truck combinations: [3]					
2, 4-tired truck	.080	12.50	.080	12.50	1.00
2, 6-tired truck	.123	8.13	.148	6.76	1.20
3	.180	5.56	.252	3.97	1.40
2-S1	.191	5.24	.267	3.75	1.40
2-S2	.217	4.61	.304	3.29	1.40
3-S2	.219	4.57	.307	3.26	1.40
2-1	.159	6.29	.223	4.48	1.40
2-2	.204	4.90	.286	3.50	1.40
2-3	.218	4.59	.305	3.28	1.40
3-3	.229	4.37	.321	3.12	1.40
3-unit, truck-tractor, semitrailer, and full trailer combination	.233	4.29	.326	3.07	1.40
All trucks, buses, and truck combinations [4]	.129	7.77	.159	6.29	1.23
All vehicles [4]	.081	12.35	.092	10.87	1.14

[1] Weighted average consumption rates developed from those used in *Supplementary Report of the Highway Cost Allocation Study* (3). Weighted averages reflect relative numbers of gasoline and diesel vehicles in each vehicle class.
[2] Total travel in rural and urban areas by each vehicle class, developed for *Highway Cost Allocation Study*, H. Doc. No. 54, 87th Cong., 1st sess., 1961, at consumption rates indicated in several studies on rural and urban operation.
[3] Each digit indicates the number of axles of a vehicle or of a unit of a vehicle combination. A single digit, or the first digit of a group symbol, represents a single-unit truck or, if followed by an S, represents a truck-tractor. The S designation represents a semitrailer. A digit without an S, in the second or third position in a group symbol, represents a full trailer.
2=2-axle single-unit truck.
3=3-axle single-unit truck.
2-S1=2-axle truck-tractor with 1-axle semitrailer.
2-S2=2-axle truck-tractor with 2-axle semitrailer.
3-S2=3-axle truck-tractor with 2-axle semitrailer.
2-1=2-axle truck with 1-axle trailer.
2-2=2-axle truck with 2-axle trailer.
2-3=2-axle truck with 3-axle trailer.
3-3=3-axle truck with 3-axle trailer.
[4] Weighted by total travel and fuel consumption of all vehicle classes indicated.

Earnings

Federal, State, and local fuel taxes, where levied, are *earned* for each mile of travel. Federal excises on tires, tubes, and tread rubber are earned in direct proportion to the amount of travel and are paid at the time the items are purchased or replenished. State and local registration fees; operator and chauffeur licenses; titling taxes; transfer, certain mileage, permit, and other fees are tax earnings in the form of annual or periodic charges.

Imposts on highway users at the local level (parking fees and other miscellaneous fees not usually considered in a user tax category, such as traffic fines and penalties, but attributable

to motor vehicles or paid as a consequence of their use) have been included with user earnings in the amounts reported received by the localities making up the SMSA's of this analysis.

Federal and State motor-fuel taxes

To obtain a consumption rate that could be applied to automobiles operating in SMSA's, consumption rates obtained for operation under different conditions in both urban and rural areas were investigated. This included consumption for automobiles operated where low average speeds and a high incidence of stop-and-go driving is necessary, and for automobiles operated where higher average speeds and fewer interruptions from traffic signals and traffic friction are possible. For example, in a report on a study made in the Philadelphia area (*4*) on the financing of road systems, a motor-fuel consumption rate 50 percent larger was used for all vehicles—automobiles and commercial vehicles—in urban areas than in rural areas. Recent studies on motor-fuel consumption rates for overall, rural, and urban vehicle operation support the evidence that fewer miles per gallon—more gallons per mile—are obtained by vehicles operated only in urban areas than by those operated only in rural areas.

Automobile motor-fuel consumption rates obtained from three studies and the rate adopted for use in this analysis are listed in table 3. The consumption rates of rural and urban operation in the first three studies were obtained from replies to questionnaires. In these studies urban operation was defined as travel at speeds of less than 35 miles per hour. The rates in table 3 for operation under rural conditions are those obtained for vehicles that were reported to have been operated 90 percent or more of their mileage at speeds of more than 35 miles per hour; the rates for urban conditions are for vehicles operated 90 percent or more of the reported mileage at speeds of less than 35 miles per hour.

The fourth set of rates was obtained in a somewhat different manner. The 0.070 gallon per mile, or 14.3 miles per gallon, rate was developed for the *Supplementary Report of the Highway Cost Allocation Study* (*3*) as a national average consumption rate for all automobiles. The rural-urban differential applied to this rate was obtained by application of estimates that reflected operating characteristics of an SMSA. Average operating speed in an urban area, the number of stops per mile, duration of stop, and average speeds on rural roads were all arbitrarily determined by considering the data collected.

After consulting with persons who analyze traffic and after reference to study data (*5*, *6*), 1½ stops per mile were used as representative of travel in an SMSA. By using measurements of fuel consumption at different speeds, while coming to a stop and accelerating again to average speed, and while idling at a stop developed in Claffey's investigation (*7*), an urban rate was obtained that was 1.08 times the average consumption rate. This ratio was applied to the 0.070-gallon-per-mile national average rate, and a resultant urban automobile consumption rate of 0.076 gallon per mile, or 13.2 miles per gallon, was obtained.

The consumption rate differential for motor-fuel used in urban areas by trucks, buses, and combinations was obtained in somewhat the same manner. The consumption rates per stop and idling time determined by Kent (*8*) and Sawhill and Firey (*9*) were used for the range of sizes and weights given for vehicles listed in table 4. The estimates—25 miles per hour average speed in an urban area, 1½ stops per mile, 15-second average idling time—were the same as those applied to automobiles. Average speeds of trucks traveling in rural areas were assumed to be 40 miles per hour; of buses, 50 miles per hour; and of automobiles, 45 miles per hour. Because most data on vehicle travel could be obtained only in the broad categories—automobiles and all other vehicles—consumption rates used were those of automobiles and the combination rate of trucks and buses.

The total motor-fuel consumption for each group of motor vehicles in each SMSA was calculated by applying the gallons-per-mile rates to the amount of travel. The Federal earnings from this motor-fuel use were obtained by multiplying the total gallonage by the Federal excise tax of 4 cents per gallon. State and local earnings from motor-fuel taxes were determined by multiplying the gallonage by the appropriate 1960 motor-fuel tax rate. Some of the tax contributions per mile of travel at the different rates at which motor fuel is taxed are, as follows:

Tax rate per gallon	*Automobile*	*Truck, bus, and truck combinations*
Cents	*Cent*	*Cent*
3	0.23	0.48
4	.30	.64
5	.38	.80
6	.45	.95
7	.53	1.11

Only the State of Missouri had a 3-cent motor-fuel tax rate in 1960; no State taxed fuel at 4 cents, which was the Federal excise tax rate. The weighted average State gasoline tax rate, nationally, was 5.92 cents per gallon (*2*, p. 2), but the weighted average tax rate for the 46 SMSA's was 5.75 cents per gallon.

The 1960 Federal Trust Fund taxes paid by highway users, other than motor-fuel taxes, are shown in table 5. An additional $5.1 million of truck, bus, and trailer excise taxes, use taxes, and rubber taxes paid on vehicles owned by the Federal Government were not included in the taxes listed in table 5 but were added to the computations in this article to obtain the rates per-vehicle-mile of travel. No distinction was made between the rural or urban rate of consumption for use taxes, except for motor fuel.

The division of vehicle excise, use, and rubber taxes between those paid for automobile use and commercial vehicle use was accomplished according to the detailed analysis prepared for *The Supplementary Report of the Highway Cost Allocation Study* (*3*). The income of the Federal Trust Fund in 1960 from taxes other than motor fuel, including payments on vehicles of Federal agencies, amounted to $450 million and was obtained from automobiles and commercial vehicles, as shown in table 6.

Table 5.—Trust Fund Taxes, 1960

Taxes	Million dollars	
Motor-fuel	------	2,269
Other	----	445
Truck, bus, and trailer excise	127	------
Motor-vehicle use	45	------
Tires, tubes, and tread rubber	273	------
TOTAL	------	2,714

State registration fees and taxes

Information about the numbers and classes of registered vehicles in the counties of an SMSA is available in different detail from the registration reports prepared by about 41 States. Also, some of the States included data by countries on payments of registration and other fees. In these States, payments were used as the total contribution by road users in the county or counties of the SMSA. In the States where such payments were not classified by counties, State per-vehicle averages for automobiles and for trucks and buses combined were multiplied by the corresponding numbers registered in the SMSA. An example of the division of receipt of State fees between automobiles and commercial vehicles is shown for Texas in table 7. As in this example, the receipts for each State were identified according to source; that is, automobiles or trucks and buses as were those of a county that had available data classified according to the vehicles for which the fees were paid. The remaining fee data were summarized, and an average per vehicle payment was obtained for each vehicle class according to the number of vehicles registered.

For an SMSA that extended beyond a State boundary, a separate computation was made so that the fee schedules of each State would be reflected. The State motor-vehicle registration and other fees obtained by these methods amounted to an *earning* of $376 million in the 46 SMSA's

Table 6.—Highway Trust Fund receipts from tire, tube, tread rubber, truck and bus vehicle-excise and vehicle-use taxes, 1960

Vehicle class for which paid	Total received	U.S.	
		Travel 1960	Estimated tax earned per mile
	Millions	*Vehicle-miles (billions)*	*Cent*
Automobile	$154	588.1	0.026
Trucks, truck combinations, and buses	296	130.7	.227
TOTAL	$450	718.8	------

Table 7.—Highway income in Texas from motor-vehicle registration and other fees, 1960

	1960	Automobiles	Trucks and buses
State registrations	4,457,022	3,534,351	922,671
State income from fees for—			
Automobile registration	$54,576,000	$54,576,000	NA
Bus registration	$474,000	NA	$474,000
Truck and truck-tractor registration	$36,884,000	NA	$36,884,000
Trailer registration	$11,516,000	NA	$11,516,000
Motorcycle registration	$220,000	$220,000	NA
Automobile operator and learner licenses	$3,830,000	$3,830,000	NA
Taxi chauffer licenses	$12,000	$12,000	NA
Bus and truck chauffeur licenses	$2,964,000	NA	$2,964,000
Other:			
Classified [1]	$1,683,000	NA	$1,683,000
Unclassified [2]	$33,248,000	$26,366,000	$6,882,000
TOTAL	$145,407,000	$85,004,000	$60,403,000
Average		$24.05	$65.47
Houston SMSA:			
Registrations	572,343	487,740	84,603
Fees	$17,269,000	$11,730,000	$5,539,000

[1] Oversize and overweight fees, carrier taxes, certificate or permit, and carrier fines and penalties, all of which are attributable to buses and trucks.

[2] Title and titling taxes, transfer, inspection, and other fees that were paid by automobiles and commercial vehicles, but no identification by whom paid was available. Division between automobiles and trucks and buses was made by prorating on the basis of the numbers of vehicles registered in the two categories.

Tolls, local taxes, and fees

Most of the local toll facilities, principally bridges, were located entirely within the 46 SMSA's. Information for this analysis was obtained from data in the financial statements of such toll facilities included in reports to Public Roads. For State-administered toll road facilities that extended beyond the boundaries of an SMSA, the State highway departments reported the total travel and the tolls earned on that travel within the SMSA. Data on local imposts on road users, and other highway income and expenditures for each SMSA, were available from the annual reports to Public Roads—see published State-local finance data (*2*, pp. 127–140). Earnings of motor-vehicle user taxes at the local, county, or city level consisted of motor-fuel taxes, motor-vehicle registration and other fees where levied, and parking fees.

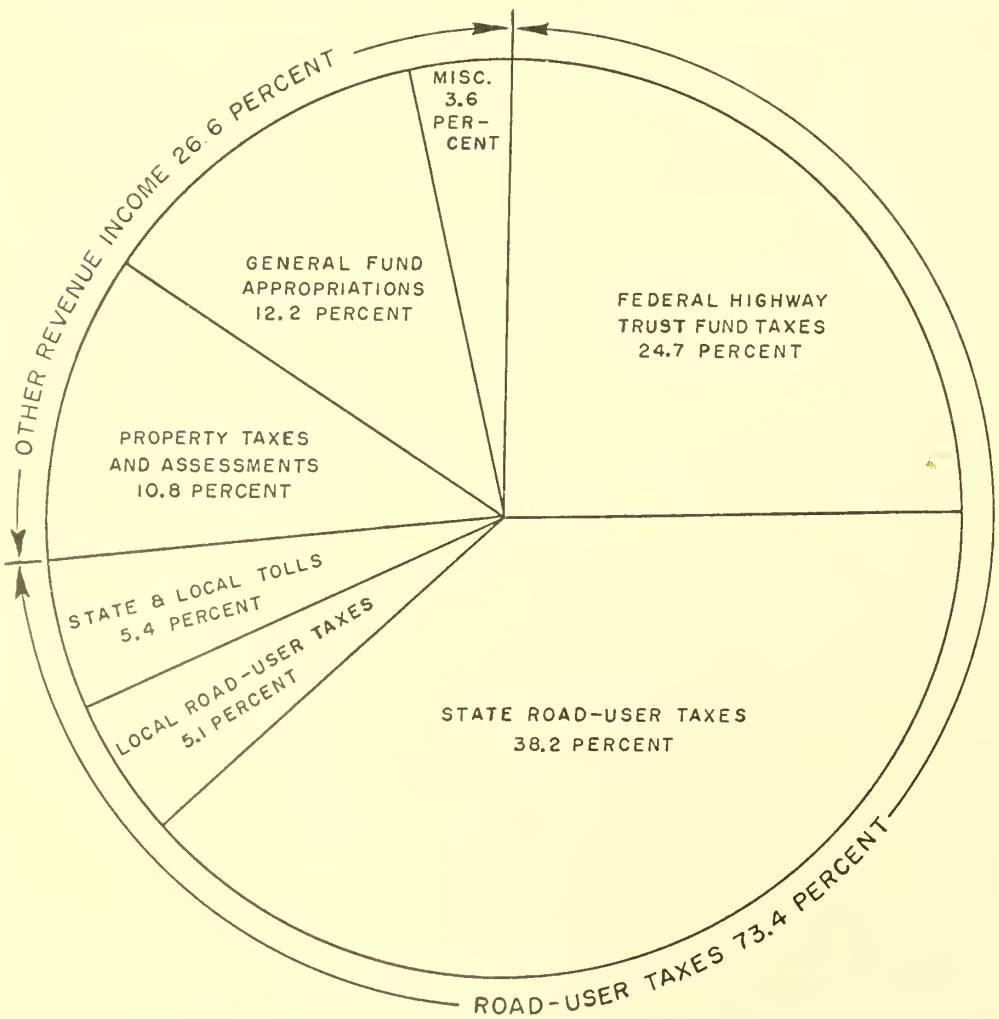
OTHER REVENUE INCOME 26.6 PERCENT
MISC. 3.6 PER-CENT
GENERAL FUND APPROPRIATIONS 12.2 PERCENT
FEDERAL HIGHWAY TRUST FUND TAXES 24.7 PERCENT
PROPERTY TAXES AND ASSESSMENTS 10.8 PERCENT
STATE & LOCAL TOLLS 5.4 PERCENT
LOCAL ROAD-USER TAXES 5.1 PERCENT
STATE ROAD-USER TAXES 38.2 PERCENT
ROAD-USER TAXES 73.4 PERCENT

Figure 2.—Sources of income for highways, 46 SMSA's, 1960.

Income for Roads and Streets

The income of each SMSA, by source—Federal, State, and local—is shown in table 8 and is classified between imposts on road users, other revenue income, and receipts from borrowing. Total income by source is illustrated proportionally in figure 2. Income from State road-user taxes equal to expenditures on State highways within each SMSA was assigned as recorded in annual reports by the States. Because Federal aid was available exclusively for capital improvements, Federal funds were assigned to each SMSA as a pro rata share of the State capital outlay in the ratio that Federal-aid reimbursements are to total capital outlay by the State. Local income, by source, is as reported annually in SMSA financial reports on roads and streets. Income from toll facilities, both State and local, is as reported for this study from annual financial reports. The total revenue income of the 46 SMSA's (table 8) amounted to $1,422 million, of which $1,044 million, or 73.4 percent, stemmed from imposts on road users and $378 million, or 26.6 percent, from property taxes and assessments, general fund appropriations, and miscellaneous sources. The income of $1,044 million from imposts on road users is 63 percent of the road-user earnings of $1,650 million in these SMSA's.

SMSA property taxes and assessments, general fund appropriations, and miscellaneous income amounted to 26.6 percent of the income for roads and streets; whereas nationally receipts from such sources (*10*) accounted for 18 percent of the total receipts for highways, including small amounts of Federal and State general fund appropriations. Investment income and borrowing of $223 million supplemented the revenue income for highways, but these items are not relevant to the comparisons made in this article. Borrowing is balanced over time by debt retirements and are not to be considered as revenue income. Investment income, a very small item, does contribute to the funds available for expenditure, but is not relevant to comparisons of user and nonuser income.

The imposts on road users, including tolls, ranged from 71 to 76 percent of the revenue income of the four SMSA groups by population size. The population group 500,000 to 1 million received the lowest percentage of its total income from road-user imposts, just under 71 percent, and it was also second lowest, 63 percent, in State and Federal road-user revenue income. The proportion of road-user tax income, excluding tolls, increases as population decreases. The road-user tax income amounted to 73 percent of the total revenue income in the smallest population group and 61 in the largest. But, the proportion of income from local user imposts and State and local tolls increases as the population increases.

Table 8.—Road and street income of 46 SMSA's, by population groups, 1960

SMSA's, by population group	Revenue income											Income from investments and borrowing	Total income
	Imposts on road users						Other				Total		
	Federal aid	State	Local (including parking fees)	Tolls		Total	Property taxes and assessments	General fund appropriations	Miscellaneous	Total			
				State facilities	Local facilities								
LESS THAN 250,000													
	Thousands	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*
Atlantic City, N.J.	$550	$1,835	$273	$380	------	$3,038	$1	$3,593	------	$3,594	$6,632	$221	$6,853
Bay City, Mich.	1,634	3,524	------	--------	------	5,158	48	449	$219	716	5,874	555	6,429
Cedar Rapids, Iowa	702	2,131	210	--------	------	3,043	2,214	22	122	2,358	5,401	561	5,962
Charleston, S.C.	1,815	2,267	------	--------	------	4,082	--------	485	------	485	4,567	--------	4,567
Eugene, Oreg.	6,727	5,780	276	--------	------	12,783	1,292	--------	39	1,331	14,114	696	14,810
Fargo, N. Dak.	8,298	4,523	------	--------	------	12,821	1,983	573	133	2,689	15,510	2,186	17,696
Fitchburg-Leominster, Mass.	53	253	1,012	--------	------	1,318	--------	269	198	467	1,785	7	1,792
Fort Wayne, Ind.	1,259	3,337	95	--------	------	4,691	1,079	171	68	1,318	6,009	--------	6,009
Great Falls, Mont.	2,052	1,465	137	--------	------	3,654	1,627	--------	------	1,627	5,281	181	5,462
Jackson, Miss.	1,858	2,048	137	--------	------	4,043	2,761	848	301	3,910	7,953	2,313	10,266
Las Vegas, Nev.	2,355	2,012	197	--------	------	4,564	635	424	139	1,198	5,762	460	6,222
Lewiston-Auburn, Maine	58	53	134	272	------	517	--------	717	15	732	1,249	20	1,269
Lexington, Ky.	880	726	81	--------	------	1,687	--------	826	------	826	2,513	188	2,701
Little Rock-North Little Rock, Ark.	12,349	6,802	188	--------	------	19,339	1,084	943	157	2,184	21,523	-------	21,523
Lynchburg, Va.	974	1,228	260	--------	------	2,462	4	412	2	418	2,880	788	3,668
Macon, Ga.	1,050	774	174	--------	------	1,998	502	493	10	1,005	3,003	-------	3,003
Madison, Wis.	3,219	6,780	360	--------	------	10,359	2,196	2,463	206	4,865	15,224	1,486	16,710
Sioux Falls, S. Dak.	4,724	3,492	158	--------	------	8,374	641	494	144	1,279	9,653	600	10,253
South Bend, Ind.	99	2,667	152	--------	------	2,918	1,039	--------	516	1,555	4,473	1,000	5,473
Springfield, Mo.	1,408	2,569	770	--------	------	4,747	860	155	265	1,280	6,027	265	6,292
Waterbury, Conn.	1,050	3,296	26	--------	------	4,372	4	1,960	287	2,251	6,623	25	6,648
Total	$53,114	$57,562	$4,640	$652	------	$115,968	$17,970	$15,297	$2,821	$36,088	$152,056	$11,552	$163,608
Percentage of revenue income	34.9	37.9	3.1	0.4	------	76.3	11.8	10.1	1.8	23.7	100.0	--------	--------
250,000 TO 500,000													
Albuquerque, N. Mex.	$6,589	$4,472	$629	--------	------	$11,690	$2,692	$52	$276	$3,020	$14,710	$2,809	$17,519
Charleston, W. Va.	422	2,220	257	--------	------	2,899	363	1,023	------	1,386	4,285	--------	4,285
Charlotte, N.C.	970	2,706	144	--------	------	3,820	--------	1,857	------	1,857	5,677	--------	5,677
Jacksonville, Fla.	10,438	10,001	427	$3,338	------	24,204	2,234	1,634	1,997	5,865	30,069	1,673	31,742
Nashville, Tenn.	11,288	6,962	1,401	--------	------	19,651	1,813	85	198	2,096	21,747	654	22,401
Omaha, Nebr.	6,591	9,489	1,987	--------	$197	18,264	5,003	--------	344	5,347	23,611	2,760	26,371
Salt Lake City, Utah	5,798	3,707	257	--------	------	9,762	2,391	943	171	3,505	13,267	--------	13,267
Tacoma, Wash.	3,341	6,495	------	--------	------	9,836	1,451	1,157	259	2,867	12,703	-------	12,703
Tulsa, Okla.	1,996	4,380	484	1,598	------	8,458	850	744	426	2,020	10,478	4,481	14,959
Wichita, Kans.	4,149	3,478	418	359	------	8,404	7,842	362	840	9,044	17,448	6,659	24,107
Wilmington, Del.	3,762	1,597	467	4,770	------	10,596	99	4,412	28	4,539	15,135	5,853	20,988
Total	$55,344	$55,507	$6,471	$10,065	$197	$127,584	$24,738	$12,269	$4,539	$41,546	$169,130	$24,889	$194,019
Percentage of revenue income	32.7	32.8	3.8	6.0	0.1	75.4	14.6	7.3	2.7	24.6	100.0	--------	--------
500,000 TO 1,000,000													
Birmingham, Ala.	$3,088	$4,020	$2,283	--------	--------	$9,391	$4,592	--------	$969	$5,561	$14,952	$3,300	$18,252
Columbus, Ohio	7,876	16,711	446	--------	--------	25,033	2,337	$696	961	3,994	29,027	8,203	37,230
Denver, Colo.	6,677	10,453	--------	$643	--------	17,773	4,441	2,580	752	7,773	25,546	47	25,593
Honolulu, Hawaii	3,806	8,284	4,368	--------	--------	16,458	3,493	85	474	4,052	20,510	--------	20,510
New Orleans, La.	8,314	8,997	539	2,926	$1,437	22,213	5,259	5,476	1,796	12,531	34,744	9,074	43,818
Phoenix, Ariz.	7,651	6,850	--------	--------	--------	14,501	1,635	5,144	4,349	11,128	25,629	4,686	30,315
Providence, R.I.	13,686	13,454	1,390	718	--------	29,248	12	10,104	222	10,338	39,586	4,368	43,954
Total	$51,098	$68,769	$9,026	$4,287	$1,437	$134,617	$21,769	$24,085	$9,523	$55,377	$189,994	$29,678	$219,672
Percentage of revenue income	26.9	36.2	4.7	2.3	0.8	70.9	11.4	12.7	5.0	29.1	100.0	--------	--------
MORE THAN 1,000,000													
Baltimore, Md.	$7,829	$32,683	$4,602	$5,558	$351	$51,023	$865	$13,016	$442	$14,323	$65,346	$4,840	$70,186
Buffalo, N.Y.	8,804	13,139	781	4,637	--------	27,361	5,069	18,223	1,277	24,569	51,930	12,774	64,704
Chicago, Ill.	88,698	104,595	39,080	18,426	2,208	253,007	30,443	5,928	2,741	39,112	292,119	83,073	375,192
Houston, Tex.	15,146	21,653	687	--------	--------	37,486	20,674	6,387	3,766	30,827	68,313	19,224	87,537
Los Angeles, Calif.	32,529	121,238	3,686	--------	281	157,734	11,202	44,776	15,262	71,240	228,974	10,720	239,694
Minneapolis-St. Paul, Minn.	26,225	27,770	1,072	--------	--------	55,067	19,945	6,832	3,175	29,952	85,019	11,495	96,514
Philadelphia, Pa.	12,406	40,074	1,999	28,034	1,566	84,079	429	26,987	7,215	34,631	118,710	14,601	133,311
Total	$191,637	$361,152	$51,907	$56,655	$4,406	$665,757	$88,627	$122,149	$33,878	$244,654	$910,411	$156,727	$1,067,138
Percentage of revenue income	21.0	39.7	5.7	6.2	0.5	73.1	9.8	13.4	3.7	26.9	100.0	--------	--------
TOTAL ALL SMSA'S	$351,193	$542,990	$72,044	$71,659	$6,040	$1,043,926	$153,104	$173,800	$50,761	$377,665	$1,421,591	$222,846	$1,644,437
Percentage of revenue income	24.7	38.2	5.1	5.0	0.4	73.4	10.8	12.2	3.6	26.6	100.0	--------	--------

(*Continued on page 194*)

Line	State	Width inches[1]	Height ft.-in.	Length-feet[2]					Number of towed units[3]			Axle load-pounds				Operating tire inflation pressure pounds per sq. in.	Pounds per engine net horsepower delivered to clutch or equivalent
				Single unit								Single		Tandem			
				Truck	Bus	Semi-trailer or trailer	Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance		
1	Alabama	96	13-6	40	40	NS	SS	NP	1	NP	NP	18,000	19,800	36,000	39,600	NS	NS
2	Alaska	96	12-6	3S	[6]40	[7]40	60	60	1	1	2	18,000		32,000		NS	N5
3	Arizona	96	13-6	40	40	N5	6S	6S	1	1	2	18,000		32,000		NS	NS
4	Arkansas	96	13-6	40	40	NS	5S	SS	1	1	NP	[8]18,000		32,000		N5	N5
5	California	96	13-6	3S	[9]3S	[7]40	60	65	NR	NR	NR	18,000		32,000		N5	NS
6	Colorado	[10]96	[11]13-6	35	40	NR	60	[12]60	1	2	2	18,000		36,000		NS	NS
7	Connecticut	102	12-6	50	50	NR	50	NP	1	NP	NP	22,400	22,848	36,000	36,720	NS	N5
8	Delaware	96	13-6	40	42	40	55	60	1	1	NP	20,000		36,000		NS	NS
9	Florida	96	13-6	[13]3S	40	[14]N5	55	5S	1	1	NP	20,000	22,000	40,000	44,000	NS	NS
10	Georgia	96	13-6	55	SS	NR	SS	55	NR	NR	NR	18,000	20,340	36,000	40,680	N5	NS
11	Hawaii	108	13-0	40	40	NR	5S	65	1	1	2	24,000		32,000		N5	N5
12	Idaho	[10]96	14-0	[18]3S	[18]3S	NR	[19]60	6S	1	1	2	[20]18,000		[20]32,000		NS	NS
13	Illinois	96	13-6	42	42	42	[21]SS	60	1	1	2	[23]18,000		32,000		NS	N5
14	Indiana	[10]96	13-6	36	40	NR	55	[21]55	1	1	2	[25]18,000	[25]19,000	[25]32,000	[25]33,000	NS	NS
15	Iowa	96	13-6	35	[6]40	[7]NR	[26]55	[26]55	1	1	2	18,000	18,540	32,000	32,960	N5	N5
16	Kansas	96	13-6	3S	[6]40	NS	S0	S0	1	1	NP	18,000		32,000		N5	NS
17	Kentucky	96	[11]13-6	[27]35	[27]35	NR	[28]55	[28]55	1	1	NP	18,000	[29]18,900	32,000	[29]33,600	NS	NS
18	Louisiana	96	13-6	35	[6]40	NR	55	60	1	1	NP	18,000		32,000		NS	NS
19	Maine	102	[31]13-6	5S	SS	NR	55	SS	1	1	NP	[31]22,000		[31]32,000		N5	NS
20	Maryland	[32]96	[33]12-6	S5	55	NR	S5	[34]5S	NR	NR	NR	22,400		[35]40,000		NS	N5
21	Massachusetts	96	NS	3S	[6]40	NR	50	NP	1	NP	NP	22,400		36,000		NS	NS
22	Michigan	96	13-6	35	40	40	[15]SS	[15]55	1	1	2	[37]18,000		[38]32,000		NS	NS
23	Minnesota	96	13-6	40	40	40	S0	50	1	1	NP	18,000		32,000		NS	N5
24	Mississippi	96	13-6	35	40	NR	S5	SS	1	1	NP	18,000		28,6S0	[22]32,000	N5	NS
25	Missouri	96	12-6	3S	40	NR	S0	S0	1	1	2	18,000		32,000		NS	NS
26	Montana	[10]96	13-6	35	40	NR	60	60	1	1	2	18,000		32,000		N5	N5
27	Nebraska	96	13-6	40	40	[39]NR	60	60	1	1	2	18,000	18,900	32,000	33,600	NS	NS
28	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	NS	N5
29	New Hampshire	96	13-6	3S	[22]40	NR	S5	SS	NP	NR	NR	22,400		36,000		N5	N5
30	New Jersey	[41]96	[41]13-6	3S	[42]3S	[7]40	5S	50	1	1	NP	22,400	23, S20	32,000	33,600	N5	NS
31	New Mexico	[43]96	13-6	40	40	NR	6S	65	1	1	2	21,600		34,320		NS	NS
32	New York	96	[33]13-0	3S	[44]3S	[7]NR	S5	5S	1	1	NP	22,400		36,000		N5	NS
33	North Carolina	96	13-6	35	[6]40	NR	55	55	1	1	NP	18,000	19,000	36,000	38,000	N5	N5
34	North Dakota	[41]96	[41]13-6	[13]3S	[6]40	NR	60	60	1	1	2	18,000		32,000		N5	NS
35	Ohio	96	13-6	3S	[6]40	[7]40	S5	60	1	NR	NR	19,000		31, S00		NS	NS
36	Oklahoma	96	13-6	3S	4S	NR	[45]50	[45]S0	1	1	NP	18,000		32,000		N5	NS
37	Oregon	96	13-6	3S	[22]40	[22]40	[21]SS	[22]6S	1	1	[22]2	[46]18,000		[46]32,000		NS	NS
38	Pennsylvania	96	[33]12-6	35	40	40	[48]50	[34]S0	1	1	NP	22,400	23,072	36,000	37,080	N5	N5
39	Rhode Island	102	12-6	40	40	40	S0	S0	1	1	NP	22,400		NS		N5	N5
40	South Carolina	96	13-6	[6]40	[6]40	NR	S5	[54]S5	1	1	NP	20,000		[55]32,000		NS	N5
41	South Dakota	96	13-6	35	40	NR	[22]65	[22]65	1	1	2	18,000		32,000		N5	N5
42	Tennessee	96	13-6	35	40	[7]N5	50	S0	1	[56]1	NP	18,000		32,000		NS	NS
43	Texas	96	13-6	35	40	NS	S0	S0	1	1	NP	18,000		32,000		N5	NS
44	Utah	96	14-0	4S	4S	45	60	60	NR	NR	NR	18,000		33,000		N5	N5
45	Vermont	96	13-6	S0	S0	NS	55	SS	1	1	NP	22,400	23,520	[57]36,000		NS	NS
46	Virginia	96	13-6	35	40	NR	50	50	1	1	NP	18,000		[58]32,000		N5	N5
47	Washington	96	13-6	3S	[6]40	40	[19]60	65	1	1	2	18,000		32,000		N5	N5
48	West Virginia	96	[33]12-6	3S	[6]40	3S	S0	S0	1	1	NP	18,000	18,900	32,000	33,600	N5	NS
49	Wisconsin	96	13-6	35	40	3S	SS	S5	1	1	NP	18,000	[60]19, S00	30,400	32,000	N5	N5
S0	Wyoming	96	13-6	40	40	NR	6S	65	1	1	2	18,000		32,000	[62]36,000	N5	NS
S1	District of Columbia	96	12-6	40	40	N5	50	50	1	1	NP	22,000		38,000		NS	NS
S2	Puerto Rico	96	12-6	35	40	NS	S0	50	1	1	NP	NS		NS		NS	N5
	AASHO Policy	102	13-6	40	40	40	S5	65	1	1	2	20,000		32,000		9S	400
Number of States	Higher	1	4	8	10	3S	14	1	6	[4]8	7	16		26		S2	52
	Same	3	37	12	37	6	24	9	46	[4]41	18	2		2S		0	0
	Lower	48	11	32	S	11	14	42	0	3	27	34		1		0	0

NP–Not permitted. NR–Not restricted. NS–Not specified.

[1] Various exceptions for farm and construction equipment, public utility vehicles, house trailers, urban, suburban, and school buses, haulage of agricultural and forest products, at wheels of vehicles for safety accessories, on designated highways, and as administratively authorized.

[2] Various exceptions for utility vehicles and loads, house trailers and mobile homes.

[3] When not specified, limited to number possible in practical combinations within permitted length limits; various exceptions for farm tractors, mobile homes, etc.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits

(1) Minimum front overhang of 3 feet, minimum spacing from first to second axle of truck tractor 8 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having S or more axles, minimum possible combined front and rear overhang assumed to be S feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerance as applicable.

[6] Less than three axles 3S feet.

[7] Trailer 3S feet.

[8] Steering axle 12,000 pounds.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 4S feet without restriction.

[10] Buses 102 inches on highways of surfaced width at least 20 feet or otherwise as administratively authorized.

[11] On class AA, or designated highways, 12 ft. 6 in. on other highways.

[12] Except 3-unit combinations may use up to 6S ft. combinations on certain highways designated by the Department of Highways.

[13] Three-axle vehicles 40 feet.

[14] Two-axle trailer 35 feet, three-axle trailer 40 feet.

[15] Auto transports permitted 63 feet.

[16] 73,280 pounds maximum, except on roads under Ru

[17] 700 (L + 40) when L is 18' or less; 800 (L + 40) wh span of 20' or over.

[18] On designated highways 40 feet.

[19] Auto transports on designated highways 65 feet.

[20] Special limits for vehicles hauling timber and timb livestock, single axle 18,900 pounds, tandem axle 37,800 p pounds maximum at 21-foot axle spacing, vehicle with 5 or

[21] 60 ft. in special cases: Illinois, auto tronsports trailers on designated major routes.

[22] On designated highways only.

[23] On designated highways; 16,000 pounds on other h

[24] Axle spacing 44 feet or more, otherwise 72,000 po

[25] On designated highways; single axle 22,400 pound of weight under one or more limitations of axle load and gr axle.

[26] Auto and boat transports and three-unit combinatio wise S0 feet for all combinations.

[27] On designated highways; trucks 26.S feet and bus

[28] State maintained highways; 45 feet on other highw

[29] Class AA highways only.

[30] Maximum gross weight on Class A highways 42,0

[31] Including load 14 feet, various exceptions for veh

[32] Vehicles loaded with tobacco hogsheads–103 inch

[33] Auto transports 13 feet 6 inches; Maryland also a flat glass.

[34] Exception for poles, pilings, structural units, row

[35] Less than 48-inch spacing, 36,000 pounds.

[36] Subject to axle and tabular limits.

[37] Single axle spaced less than 9 feet from nearest

OTOR VEHICLES COMPARED WITH AASHO STANDARDS

u of Public Roads

, 1964

ss weight limit			Specified maximum gross weight-pounds[4]						Practical maximum gross weight-pounds[5]						
	Applicable to:		Truck		Truck-tractor semitrailer				Truck		Truck-tractor semitrailer				
of ction	Any group of axles	Total wheel-base only	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	Line
		X					73,280		27,800	47,600	47,600	67,400	73,280	NP	1
cap.	Under 18'	Over 18'	29,000	43,000	47,000	61,000	75,000	76,800	26,000	40,000	44,000	58,000	72,000	76,800	2
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	3
mum									26,000	40,000	44,000	58,000	72,000	73,280	4
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	5
ec. lim.		X							26,000	44,000	44,000	62,000	76,000	76,000	6
tire cap.			32,000	53,800	53,800	67,400	73,000	NP	30,848	44,720	53,800	67,400	73,000	NP	7
. lim.	X		30,000	46,000	48,000				28,000	44,000	48,000	64,000	73,280	73,280	8
		X							30,000	52,000	52,000	73,271	73,271	73,271	9
[16]							73,280	73,280	28,340	48,680	48,680	69,110	73,280	73,280	10
	X								32,000	40,000	56,000	64,000	72,000	80,000	11
	X								26,000	40,000	44,000	58,000	73,280	76,800	12
tire cap.			36,000	50,000	50,000	64,000	[24]73,280	[24]73,280	26,000	40,000	44,000	58,000	72,000	73,280	13
tire cap.			36,000	50,000	54,000			72,000	27,000	41,000	45,000	59,000	[25]73,000	[25]73,000	14
	X								26,540	40,960	45,080	59,500	73,280	73,280	15
		X							26,000	40,000	44,000	58,000	72,000	73,280	16
tire cap [30]			27,000	42,000	42,000	59,640	73,280	NP	27,000	42,000	42,000	59,640	73,280	73,280	17
re cap.									26,000	40,000	44,000	58,000	72,000	76,000	18
cap.	X		32,000	[31]51,800	51,800	62,050	73,280	73,280	30,000	40,000	51,800	62,000	72,000	73,280	19
		X			[36]55,000	[36]65,000	[36]73,280	[36]73,280	30,400	48,000	52,800	55,000	73,280	73,280	20
. lim.		X	[36]46,000	[36]60,000	[36]60,000	[36]73,000	[36]73,000	NP	30,400	44,000	52,800	66,400	73,000	NP	21
re cap.									26,000	[22]40,000	44,000	[22]58,000	[22]66,000	[22]102,000	22
	X						73,280	73,280	26,000	40,000	44,000	58,000	72,000	73,280	23
cap.	X								26,000	[22]40,000	44,000	58,000	[22]72,000	[22]73,280	24
	X								26,000	40,000	44,000	58,000	[22]72,000	[22]73,280	25
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	26
	X		36,000	54,000	54,000	71,146	71,146	71,146	26,780	41,200	45,320	59,740	73,280	73,280	27
	Under 18'	Over 18'							26,900	41,600	45,800	60,500	75,200	76,800	28
. lim.		X	33,400	[40]55,000	[40]52,800	66,400	73,280	73,280	30,400	44,000	52,800	66,400	73,280	73,280	29
ire cap.									31,520	41,600	55,040	65,120	73,280	73,280	30
	Under 18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	31
		X					71,000	71,000	30,400	44,000	52,800	66,400	71,000	71,000	32
			31,500	49,875	49,875	67,200	73,280	73,280	27,000	46,000	46,000	65,000	73,280	73,280	33
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	73,280	34
		X							27,000	39,500	46,000	[58],500	71,000	78,000	35
	X								26,000	40,000	44,000	58,000	72,000	73,280	36
	Under 18'	Over 18'					[47]76,000	[47]76,000	26,000	40,000	44,000	58,000	72,000	[47]76,000	37
[49]			33,000	47,000	50,000	60,000	71,145	71,145	31,072	45,080	51,500	61,800	73,280	73,280	38
			[50]36,000	[51]44,000	[52]53,800	[53]67,400	73,280	88,000	30,400	44,000	53,800	67,400	73,280	88,000	39
			32,000	46,000	50,000	65,000	73,280	73,280	28,000	40,000	48,000	60,000	72,000	73,280	40
	X								26,000	40,000	44,000	58,000	72,000	73,280	41
		X		44,000	48,000	62,000	73,280		26,000	40,000	44,000	58,000	72,000	43,500	42
	X								26,000	40,000	44,000	58,600	72,000	72,000	43
	X		36,000	51,000	54,000	69,000	79,900	79,900	26,000	41,000	44,000	59,000	74,000	79,900	44
cap.	X						73,280	73,280	31,520	44,000	55,000	66,400	73,280	73,280	45
		X					70,000	70,000	26,000	40,000	44,000	60,000	70,000	70,000	46
	Under 18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	47
	X					[59]70,000	[59]70,000	[59]70,000	26,900	41,600	45,800	60,500	73,280	73,280	48
	X								27,500	40,000	47,000	59,500	73,000	73,000	49
	X								26,000	44,000	44,000	62,000	73,950	73,950	50
cap.	X						70,000	70,000	30,000	46,000	52,000	68,000	70,000	70,000	51
tire cap.															52
	X								28,000	40,000	48,000	60,000	72,000	86,500	
5									15	25	15	22	25	2	
33	18	20							2	24	2	3	20	0	
14									34	2	34	26	6	49	

nority 56,000 pounds maximum.
than 18'; 900(L + 40) on highways having no structures with

s, concentrates, aggregates, and agricultural products including
weight table: vehicle with 3 or 4 axles permitted 66,000
rmitted 79,000 pounds maximum at 43-foot axle spacing.
rucks pulling house trailers only, Oregon, truck tractor semi-

6,000 pounds; tolerance of 1,000 pounds on total of all excesses
pending upon the placing of 9000 # on the front or steering
0 feet on highways with surface width 22 feet or more; other-
ther highways.

lass B highways 30,000 pounds.
orest products and construction materials.
inches for vehicles loaded with hoy or strow, or carrying
, permitted 70 feet.

3,000 pounds.

[38] On designated highways only and limited to one tandem axle in combination, otherwise 26,000 pounds.
[39] Trailer 40 feet.
[40] On Interstate System 47,500 pounds.
[41] Vehicles in excess may be operated under special permit obtained in advance, in New Jersey from the Department of Motor Vehicles; in North Dakota, from State Highway Truck Regulatory Department.
[42] Or as prescribed by P.U.C.
[43] On designated highways 102 inches. Body restricted to 96", additional 6" for tires only.
[44] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.
[45] Auto transports, oil field equipment, by special permit only, 60 feet.
[46] Lagging vehicles permitted 7-foot wheelbase tolerance, 19,000-single axle, 34,000-pounds tandem axle.
[47] Governs gross weight permitted on highways designated by resolution of State highway commission.
[48] Where truck-tractor was properly registered in Pennsylvania as of December 31, 1961, 55 feet.
[49] Single unit truck with 4 axle permitted 60,000 pounds.
[50] Axles spaced less than 6 feet 32,000 pounds, less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.
[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds, less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.
[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 53,800 pounds.
[53] Legal limit 67,400 pounds, axle spacing 27 feet or more.
[54] House trailers, auto transports, and double saddle mounts in daylight hours, 60 feet.
[55] On Interstate System, 36,000 pounds on other roads.
[56] Limited to 3,500 pounds.
[57] Three-axle tandem 42,700 pounds.
[58] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.
[59] Only on certain highways, or portions thereof, designated by State Roads Commissioner, and consistent with Congressional action.
[60] Axle load 21,000 pounds on 2-axle trucks hauling peeled or unpeeled forest products cut crosswise or transporting milk from farm to market but not over Interstate System.
[61] On Class A highways. All axles of a vehicle or combination—73,000 pounds maximum. Wheel, axle, axle group and gross vehicle weights on Class B highways are 60% of weights including tolerance authorized for Class A highways.
[62] Based on ruling of Attorney General.

(*Continued from page 191*)

Table 9.—Expenditures for roads and streets in 46 SMSA's, 1960

SMSA's, by population group	Capital outlay [1]							Maintenance, administration, operation, etc.[2]	Interest	Total expenditures	Debt retirement	Total disbursements
	On State-administered highways			On local roads and streets			Total					
	Rural	Municipal	Total	Rural	Municipal	Total						
LESS THAN 250,000												
	Thousands	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*
Atlantic City, N.J.	$1,044	$51	$1,095	$165	$293	$458	$1,553	$4,608	$298	$6,459	$406	$6,865
Bay City, Mich.	3,773	99	3,872	41	745	786	4,658	1,614	14	6,286	90	6,376
Cedar Rapids, Iowa	907	370	1,277	888	911	1,799	3,076	2,552	56	5,684	279	5,963
Charleston, S.C.	2,817	353	3,170	99	84	183	3,353	1,334	------	4,687	--------	4,687
Eugene, Oreg.	7,849	--------	7,849	1,763	1,135	2,898	10,747	3,084	54	13,885	246	14,131
Fargo, N. Dak.	10,265	985	11,250	1,572	1,552	3,124	14,374	2,361	273	17,008	536	17,544
Fitchburg-Leominster, Mass.	99	--------	99	46	206	252	351	1,289	12	1,652	140	1,792
Fort Wayne, Ind.	1,952	205	2,157	809	599	1,408	3,565	1,875	97	5,537	307	5,844
Great Falls, Mont.	2,834	70	2,904	42	241	283	3,187	1,443	241	4,871	475	5,346
Jackson, Miss.	2,287	478	2,765	505	1,033	1,538	4,303	2,598	295	7,196	2,485	9,681
Las Vegas, Nev.	3,011	24	3,035	180	853	1,033	4,068	1,830	41	5,939	234	6,173
Lewiston-Auburn, Maine	4	110	114	--------	110	110	224	795	199	1,218	42	1,260
Lexington, Ky.	1,492	73	1,565	--------	--------	[3]	1,565	1,138	--------	2,703	--------	2,703
Little Rock-North Little Rock, Ark.	4,739	12,698	17,437	99	1,530	1,629	19,066	2,964	141	22,171	345	22,516
Lynchburg, Va.	1,243	168	1,411	--------	604	604	2,015	1,229	120	3,364	304	3,668
Macon, Ga.	1,559	--------	1,559	345	135	480	2,039	915	24	2,978	45	3,023
Madison, Wis.	6,127	166	6,293	2,006	3,129	5,135	11,428	3,914	227	15,569	1,069	16,638
Sioux Falls, S. Dak.	4,428	2,782	7,210	1,004	241	1,245	8,455	1,539	14	10,008	20	10,028
South Bend, Ind.	153	16	169	477	436	913	1,082	2,800	25	3,907	575	4,482
Springfield, Mo.	2,911	642	3,553	67	307	374	3,927	1,656	29	5,612	330	5,942
Waterbury, Conn.	3,216	--------	3,216	104	245	349	3,565	3,089	103	6,757	192	6,949
Total	$62,710	$19,290	$82,000	$10,212	$14,389	$24,601	$106,601	$44,627	$2,263	$153,491	$8,120	$161,611
Percentage of expenditures	40.8	12.6	53.4	6.6	9.4	16.0	69.4	29.1	1.5	100.0	--------	--------
250,000 TO 500,000												
Albuquerque, N. Mex.	$1,031	$9,157	$10,188	$101	$2,268	$2,369	$12,557	$3,726	$469	$16,752	$1,492	$18,244
Charleston, W. Va.	655	--------	655	--------	37	37	692	3,426	116	4,234	176	4,410
Charlotte, N.C.	1,436	396	1,832	--------	428	428	2,260	3,140	181	5,581	298	5,879
Jacksonville, Fla.	17,862	6,338	24,200	143	1,058	1,201	25,401	7,898	5,011	38,310	6,963	45,273
Nashville, Tenn.	7,465	8,694	16,159	626	1,126	1,752	17,911	3,454	276	21,641	866	22,507
Omaha, Nebr.	8,205	2,740	10,945	3,156	3,708	6,864	17,809	6,510	402	24,721	2,875	27,596
Salt Lake City, Utah	7,931	174	8,105	778	685	1,463	9,568	3,477	--------	13,045	--------	13,045
Tacoma, Wash.	2,044	4,629	6,673	--------	2,795	2,795	9,468	3,736	179	13,383	346	13,729
Tulsa, Okla.	3,347	419	3,766	1,739	1,856	3,595	7,361	5,151	1,757	14,269	1,547	15,816
Wichita, Kans.	3,658	2,894	6,552	1,152	4,228	5,380	11,932	4,637	1,330	17,899	7,116	25,015
Wilmington, Del.	6,342	1,958	8,300	48	943	991	9,291	6,336	1,145	16,772	3,246	20,018
Total	$59,976	$37,399	$97,375	$7,743	$19,132	$26,875	$124,250	$51,491	$10,866	$186,607	$24,925	$211,532
Percentage of expenditures	32.2	20.0	52.2	4.2	10.2	14.4	66.6	27.6	5.8	100.0	--------	--------
500,000 TO 1,000,000												
Birmingham, Ala.	$4,988	$103	$5,091	$2,718	$2,251	$4,969	$10,060	$4,144	$217	$14,421	$846	$15,267
Columbus, Ohio	7,697	9,822	17,519	1,543	1,892	3,435	20,954	7,196	829	28,979	8,774	37,753
Denver, Colo.	5,920	5,033	10,953	1,625	1,677	3,302	14,255	10,438	325	25,018	455	25,473
Honolulu, Hawaii	2,877	4,584	7,461	------	3,334	3,334	10,795	7,801	475	19,071	446	19,517
New Orleans, La.	4,051	12,478	16,529	1,693	3,594	5,287	21,816	9,892	5,627	37,335	5,638	42,973
Phoenix, Ariz.	2,917	6,046	8,963	8,153	3,117	11,270	20,233	6,767	440	27,440	1,408	28,848
Providence, R.I.	3,197	20,518	23,715	459	4,372	4,831	28,546	12,793	459	41,798	2,038	43,836
Total	$31,647	$58,584	$90,231	$16,191	$20,237	$36,428	$126,659	$59,031	$8,372	$194,062	$19,605	$213,667
Percentage of expenditures	16.3	30.2	46.5	8.4	10.4	18.8	65.3	30.4	4.3	100.0	------	--------
MORE THAN 1,000,000												
Baltimore, Md.	$10,812	$37	$10,849	$4,146	$15,761	$19,907	$30,756	$27,208	$4,245	$62,209	$8,322	$70,531
Buffalo, N.Y.	10,100	9,193	19,293	4,346	6,806	11,152	30,445	22,304	3,719	56,468	10,455	66,923
Chicago, Ill.	46,414	120,089	166,503	8,573	26,159	34,732	201,235	101,616	31,450	334,301	32,764	367,065
Houston, Tex.	--------	35,954	35,954	2,548	12,087	14,635	50,589	16,949	4,403	71,941	10,190	82,131
Los Angeles, Calif.	38,885	59,884	98,769	15,494	31,647	47,141	145,910	78,583	1,692	226,185	4,548	230,733
Minneapolis-St. Paul, Minn.	7,923	33,772	41,695	6,046	11,761	17,807	59,502	27,096	1,511	88,109	5,729	93,838
Philadelphia, Pa.	23,110	20,642	43,752	3,512	12,195	15,707	59,459	46,787	11,259	117,505	16,506	134,011
Total	$137,244	$279,571	$416,815	$44,665	$116,416	$161,081	$577,896	$320,543	$58,279	$956,718	$88,514	$1,045,232
Percentage of expenditures	14.4	29.2	43.6	4.7	12.1	16.8	60.4	33.5	6.1	100.0	- ----	--------
TOTAL, All SMSA's	$291,577	$394,844	$686,421	$78,811	$170,174	$248,985	$935,406	$475,692	$79,780	$1,490,878	$141,164	$1,632,042
Percentage of expenditures	19.5	26.5	46.0	5.3	11.4	16.7	62.7	31.9	5.4	100.0	------	--------

[1] In some SMSA's, the classification of expenditures by system is not exact. In the Baltimore area for example, the State and Federal-aid expenditures for municipal extensions of State highways are included with local municipal street expenditures because State highways stop at the Baltimore municipal limits. When the expenditures for rural and municipal State highways were not segregated, the amounts are given under rural.

[2] Includes parking, policing, and allied street functions.

[3] No local capital outlay given in the report of expenditures for 1960.

Expenditures

Expenditures for highways are detailed for capital outlays between those expended on State administered highways and those on local roads and streets, as shown in table 9 and figure 3. Because of the rural areas in the SMSA's, rural-municipal classification is shown to the extent that the data permitted. The classification of expenditures for construction, by systems, is frequently not complete or exact, or the rural-municipal segregation of expenditures on State-administered highways may not be complete. In the Baltimore, Md., Area, for example—perhaps the most extreme—Federal aid of $7.8 million, matched with $5.5 million of local municipal funds, was expended for construction of Federal-aid route extensions in the city of Baltimore that ordinarily would be extensions of State highways. But because State jurisdiction stops at the Baltimore city limit, such extensions are under municipal jurisdiction. The capital expenditure of the entire $13.4 million is included in table 9 with those for local municipal streets rather than with those for municipal extensions of State-administered systems, as for the other Areas.

The split between rural and municipal expenditures is further complicated because the State classification of rural expenditures includes Federal-aid urban outlay to the extent that the Federal-aid urban area extends beyond the corporate limits of cities. Although such classification difficulties affect the columnar data of individual SMSA's in table 9, the total outlays shown are complete.

The total 1960 expenditures for highways were $1,491 million in the 46 SMSA's. Capital outlay, which consists of right-of-way, engineering, and construction costs, amounted to $935 million, 62.7 percent of total expenditures. Maintenance, operation, and administration expenditures were $476 million, or 31.9 percent, and interest and financing costs the remainder, $80 million or 5.4 percent.

The broad classification of maintenance, administration, and operation requires some definition. Local expenditures were considered to be complete and adequately reported. Expenditures for maintenance and administration of local toll facilities were equally well reported. At the State level, maintenance of condition and operating maintenance were included in the analysis for this study. But expenditures for administration, collection, and State highway police and safety were not included in this analysis because proration would be required that might be misleading; nevertheless, earnings from highway use are expended for these functions.

The magnitude of the cost of collecting user taxes, administration of highway organizations, and highway police and safety expenditures of the States in 1960 is illustrated by the following figures. For example, the total for highway administration, collection of user taxes, and highway police and safety were, respectively, for all States and the District of Columbia: $290 million, $212 million, and $234 million. The same expenditures prorated to the 46 SMSA's were: $35 million, $36 million, and $39 million. Costs of collection,

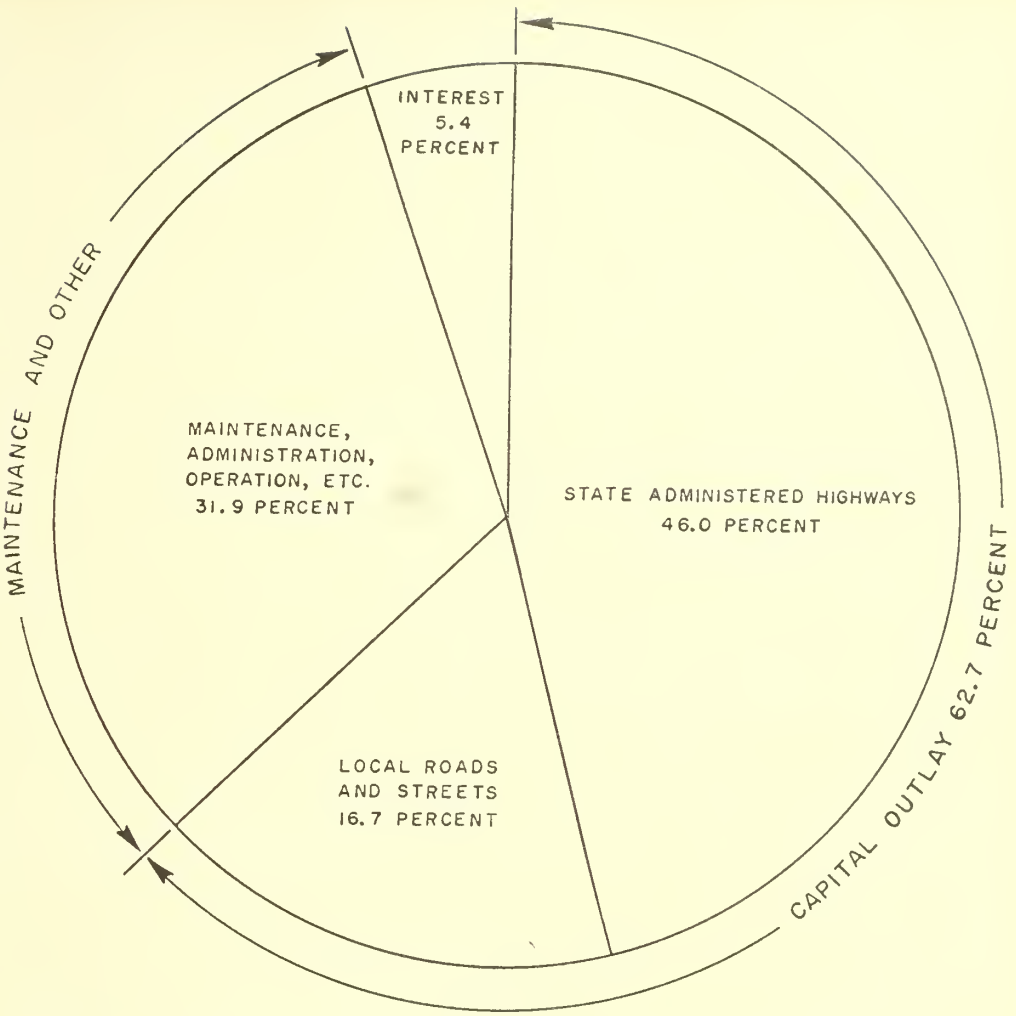

Figure 3.—Expenditures for highways, 46 SMSA's, 1960.

including administrative and enforcement costs, were obtained by relating the total State costs to total State user revenues in each State and applying that relationship to State user-tax earnings of the SMSA in that State. State highway administration costs were related to total capital and maintenance expenditures in each State, and that relationship was applied to State expenditures for capital outlay and maintenance of each SMSA. Expenditures for State highway police and safety education were derived only in total for all SMSA's by relating all travel in the SMSA's to the total travel nationwide.

Table 10.—Disbursements for allied street functions and parking facilities for the United States and 46 SMSA's, 1960 [1]

	Allied street functions					Parking facilities	Total
	Street lighting	Street cleaning	Sidewalks	Storm sewers	Total		
United States:	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*
Capital outlay	$33,615	$2,853	$19,277	$87,393	$143,138	$55,757	$198,895
Maintenance and operation	176,559	67,290	4,425	14,651	262,925	37,093	300,018
Interest on debt	[2]	[2]	[2]	[2]	11,648	13,694	25,342
Total expenditures					417,711	106,544	524,255
Fund transfers to—							
Municipal street funds						73,974	73,974
Allied street functions						2,310	2,310
County road funds						523	523
Other purposes					6,977	5,142	12,119
Total					$6,977	$81,949	$88,926
Debt retirement	[2]	[2]	[2]	[2]	$31,499	$18,658	$50,157
TOTAL DISBURSEMENTS					$456,187	$207,151	$663,338
SMSA's (46):							
Total expenditures	$59,111	$19,449	$2,876	$36,647	$118,083	$16,370	$134,453

[1] Source: *Highway Statistics 1961*, Bureau of Public Roads, 1963, table UF-12-1960, p. 79.
[2] Debt service for these functions grouped in total, no breakdown available.

Table 11.—Estimated earnings of motor-vehicle-user taxes and tolls generated by travel and vehicle ownership in 46 SMSA's in 1960, at 1960 tax rates and tolls

SMSA's, by population group	Vehicle-miles of travel within the SMSA	Collecting agencies							Total user taxes on highway use in SMSA	Ratio of user earnings to expenditures (Expenditures=1.0)
		Federal Government	State agencies [1]			Local governments [1]				
		Excise taxes of the Federal Highway Trust Fund [2]	Motor-fuel and vehicle taxes and fees	Tolls	Total State taxes and fees	Motor-fuel and vehicle taxes and fees [3]	Tolls	Total local taxes and fees		
LESS THAN 250,000										
	Million	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*	
Atlantic City, N.J	1,175	$4,690	$7,102	$380	$7,482	$273	--------	$273	$12,445	1.93
Bay City, Mich	425	1,751	3,354	--------	3,354	----------	--------	----------	5,105	.81
Cedar Rapids, Iowa	502	2,125	4,988	--------	4,988	210	--------	210	7,323	1.29
Charleston, S.C	715	3,103	5,332	--------	5,332	----------	--------	----------	8,435	1.80
Eugene, Oreg	643	2,597	6,456	--------	6,456	276	--------	276	9,329	.67
Fargo, N. Dak	452	1,962	3,835	--------	3,835	----------	--------	----------	5,797	.34
Fitchburg-Leominster, Mass	426	1,492	2,286	--------	2,286	1,012	--------	1,012	4,790	2.90
Fort Wayne, Ind	582	2,399	5,072	--------	5,072	95	--------	95	7,566	1.37
Great Falls, Mont	273	1,048	2,174	--------	2,174	137	--------	137	3,359	.69
Jackson, Miss	590	2,526	5,112	--------	5,112	137	--------	137	7,775	1.08
Las Vegas, Nev	359	1,722	3,658	--------	3,658	197	--------	197	5,577	.94
Lewiston-Auburn, Maine	174	622	1,650	272	1,922	134	--------	134	2,678	2.20
Lexington, Ky	433	1,832	3,666	--------	3,666	81	--------	81	5,579	2.06
Little Rock-North Little Rock, Ark	796	3,876	7,330	--------	7,330	188	--------	188	11,394	.51
Lynchburg, Va	424	1,837	3,220	--------	3,220	260	--------	260	5,317	1.58
Macon, Ga	452	1,721	3,273	--------	3,273	174	--------	174	5,168	1.74
Madison, Wis	911	3,916	7,436	--------	7,436	360	--------	360	11,712	.75
Sioux Falls, S. Dak	340	1,435	3,027	--------	3,027	158	--------	158	4,620	.46
South Bend, Ind	570	2,301	4,888	--------	4,888	152	--------	152	7,341	1.88
Springfield, Mo	659	2,505	2,962	--------	2,962	770	--------	770	6,237	1.11
Waterbury, Conn	523	2,146	4,208	--------	4,208	26	--------	26	6,380	.94
Total	11,424	$47,606	$91,029	$652	$91,681	$4,640	--------	$4,640	$143,927	.94
Percentage of user taxes	----------	33.1	63.3	0.4	63.7	3.2	--------	3.2	100.0	----------
250,000 TO 500,000										
Albuquerque, N. Mex	828	$3,531	$7,109	--------	$7,109	$629	--------	$629	$11,269	0.67
Charleston, W. Va	815	3,504	8,734	--------	8,734	257	--------	257	12,495	2.95
Charlotte, N.C	675	2,741	6,462	--------	6,462	144	--------	144	9,347	1.67
Jacksonville, Fla	1,807	7,297	15,715	$3,338	19,053	427	--------	427	26,777	.70
Nashville, Tenn	1,209	5,147	10,389	--------	10,389	1,401	--------	1,401	16,937	.78
Omaha, Nebr	1,842	7,585	14,988	--------	14,988	1,987	$197	2,184	24,757	1.00
Salt Lake City, Utah	1,155	4,929	8,908	--------	8,908	257	--------	257	14,094	1.08
Tacoma, Wash	1,281	5,087	12,035	--------	12,035	----------	--------	----------	17,122	1.28
Tulsa, Okla.[4]	1,436	5,931	15,757	1,598	17,355	484	--------	484	23,770	1.67
Wichita, Kans	1,382	5,514	8,846	359	9,205	418	--------	418	15,137	.85
Wilmington, Del	1,586	6,554	11,000	4,770	15,770	467	--------	467	22,791	1.36
Total	14,016	$57,820	$119,943	$10,065	$130,008	$6,471	$197	$6,668	$194,496	1.04
Percentage of user taxes	----------	29.7	61.7	5.2	66.9	3.3	0.1	3.4	100.0	----------
5,000,000 TO 1,000,000										
Birmingham, Ala	2,052	$8,140	$14,661	--------	$14,661	$2,283	--------	$2,283	$25,084	1.74
Columbus, Ohio	2,696	10,308	23,291	--------	23,291	446	--------	446	34,045	1.17
Denver, Colo	3,500	14,354	26,939	$643	27,582	----------	--------	----------	41,936	1.68
Honolulu, Hawaii	1,123	4,544	8,903	--------	8,903	4,368	--------	4,368	17,815	.93
New Orleans, La	1,941	9,138	16,690	2,926	19,616	539	$1,437	1,976	30,730	.82
Phoenix, Ariz	3,084	13,157	21,430	--------	21,430	----------	--------	----------	34,587	1.26
Providence, R.I	3,401	11,997	27,150	718	27,868	1,390	--------	1,390	41,255	.99
Total	17,797	$71,638	$139,064	$4,287	$143,351	$9,026	$1,437	$10,463	$225,452	1.16
Percentage of user taxes	----------	31.8	61.7	1.9	63.6	4.0	0.6	4.6	100.0	----------
MORE THAN 1,000,000										
Baltimore, Md	5,966	$24,796	$48,698	$5,558	$54,256	$4,602	$351	$4,953	$84,005	1.35
Buffalo, N.Y	3,417	13,638	31,416	4,637	36,053	781	--------	781	50,472	.89
Chicago, Ill	19,210	76,656	135,788	18,426	154,214	39,080	2,208	41,288	272,158	.81
Houston, Tex	4,265	17,958	36,935	--------	36,935	687	--------	687	55,580	.77
Los Angeles, Calif	27,808	117,625	264,876	--------	264,876	3,686	281	3,967	386,468	1.71
Minneapolis-St. Paul, Minn	5,500	22,557	42,210	--------	42,210	1,072	--------	1,072	65,839	.75
Philadelphia, Pa	12,314	48,908	90,797	28,034	118,831	1,999	1,566	3,565	171,304	1.46
Total	78,480	$322,138	$650,720	$56,655	707,375	$51,907	$4,406	$56,313	$1,085,826	1.13
Percentage of user taxes	----------	29.7	59.9	5.2	65.1	4.8	0.4	5.2	100.0	----------
TOTAL, ALL SMSA'S	121,717	$499,202	$1,000,756	$71,659	$1,072,415	$72,044	$6,040	$78,084	$1,649,701	1.11
Percentage of user taxes	----------	30.2	60.7	4.3	65.0	4.4	0.4	4.8	100.0	----------

[1] Includes earnings from State motor-fuel taxes at estimated consumption rates per mile of travel, and registration, operator license, and other fees that were either recorded collections in each area, or computed on basis of vehicle ownership in that SMSA. Local highway-user imposts include proceeds from motor fuel, bus and wheel taxes, automobile and truck licenses, and other fees levied on highway users within those jurisdictions.

[2] Includes taxes on motor fuel, truck, bus, and trailer excise, tires, tubes, and tread rubber, and vehicle-use taxes. Does not include automobile excise, parts and accessories, and lubricating oil taxes that are general fund revenues.

[3] Includes parking fees.

[4] Excludes Osage County.

Allied street functions

As stated previously, charges for State administration, collection, and police and safety have not been included with the expenditures shown in table 9. However, expenditures for indirect municipal street functions, usually termed *allied street functions*, have been included in the amounts shown for maintenance and administration. These allied street functions consist of construction, maintenance, and administration for street lighting, street cleaning, sidewalks, storm drainage, and maintenance and operation of parking meters and lots. Because division of the expenditures for construction or maintenance was not always reported, the entire expenditure for allied street function has been included in the amounts shown in table 9 for maintenance and administration.

Except for parking facilities, the expenditures for allied street functions provide the most benefits to abutting property and in the form of protection of public health, safety, and security. For example, storm sewers drain adjacent property as well as the streets, and street lighting provides safety and security to pedestrians and occupants of nearby buildings. Inclusion of such expenditures in the statistical data presented here may therefore be regarded as an offset against the omission of the allocated expenditures for State highway administration, user-tax revenue collection, and State highway police.

Funds for allied street functions are derived mainly from property taxes and assessments and from local general revenue funds. Road-user revenues, State and local, are used to some extent when outlays for these purposes are incidental to highway construction or maintenance operations. The amounts shown in table 8 include the income from these different sources for the indirect municipal street functions, and the expenditures amounting to $134 million are included with the amounts shown in table 9. A summary of the nationwide expenditures for allied street functions in 1960 and the corresponding expenditures in the 46 SMSA's are shown in table 10.

Road-user earnings

The motor-vehicle user tax and toll earnings for each SMSA are listed in table 11, and figure 4 shows the proportion of the total earnings, by Federal, State, and local sources. The proportions of earnings from Federal, State, and local levies are given in percentages. State motor-fuel taxes and motor-vehicle registration and other fees were 60.7 percent of total earnings, double those realized from the Federal excise group. Local levies on road users amounted to 4.4 percent of the total and are equal to the earnings of the State level toll facilities. Tolls from all facilities, State and local, were $79 million, or 4.7 percent.

The total road-user tax earnings of $1,650 million divided by the number of vehicle-miles equals a payment of 1.36 cents per vehicle-mile of travel; of this amount, 0.41 cent was from Federal Trust Fund excise taxes and 0.88 cent from State taxes and tolls. Local user levies amounted to an average per vehicle-mile earnings of slightly less than 0.07 cent.

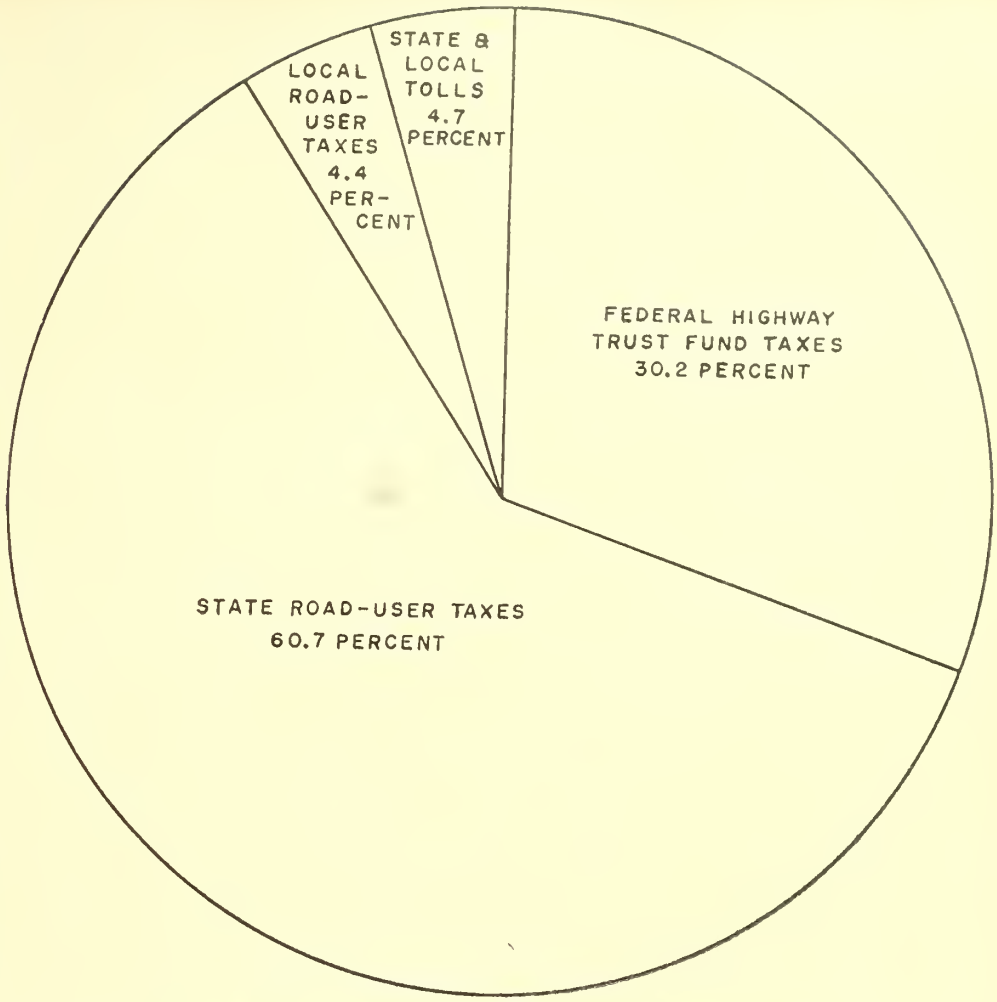
LOCAL ROAD-USER TAXES 4.4 PER-CENT
STATE & LOCAL TOLLS 4.7 PERCENT
FEDERAL HIGHWAY TRUST FUND TAXES 30.2 PERCENT
STATE ROAD-USER TAXES 60.7 PERCENT

Figure 4.—Estimated earnings for highways, 46 SMSA's, 1960.

By comparison, the total 1960 U.S. road-user taxes (*10*) amounted to $8,211 million, and the corresponding total travel (*2*, p. 80) was 718,845 million vehicle-miles. These two items equal an average earning rate of 1.14 cents per vehicle-mile of travel. The computed earning rate of 1.36 cents per vehicle-mile of travel for the SMSA's was 19 percent more than the national average. The differences in the motor-fuel consumption rate estimated for the SMSA's accounted for 14 of the 19 percent earning rate (table 4). The remainder of the difference is accounted for by the fact that the annual travel in the SMSA per vehicle registered therein (a synthetic figure because much of the travel is contributed by vehicles from outside the Area) runs low—8,497 miles per year, in comparison with the national per vehicle average, which was 9,652 in 1960. When the annual mileage is low the effect of registration and other flat fees causes the payment per vehicle-mile to be high.

The last column of table 11 shows the ratio of motor-vehicle user tax earnings to total current highway expenditures in 1960 as shown in table 9. In 26 of the 46 SMSA's, earnings of user taxes were more than the total current expenditures for highways. The ratios for the several population groups differ, generally upward as population increases, but there is no similar consistency in the number of places that have ratios of earnings to expenditures of more or less than 1.00, as shown in table 12.

Statistical Summary

The information in tables 8, 9, and 11 is summarized in table 13, from which a comparison of road and street income, expenditures, and road-user earnings of each SMSA population group can be made. In the 46

Table 12.—Ratio of earnings to expenditures, by population groups

Population group	Ratio of earnings to expenditures	
	SMSA's ratios larger than 1.0	Group ratio
	Number	*Percent*
Less than 250,000	12 out of 21	0.94
250,000–500,000	7 out of 11	1.04
500,000–1,000,000	4 out of 7	1.16
More than 1,000,000	3 out of 7	1.13
All SMSA's	26 out of 46	1.11

Table 13.—Highway income, expenditures, and earnings of motor-vehicle-user taxes, 46 SMSA's 1960

	Standard Metropolitan Statistical Area population group—									
	Less than 250,000		250,000 to 500,000		500,000 to 1,000,000		1,000,000 and more		Totals, 46 SMSA's	
INCOME										
Imposts on highway users:	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*
State and Federal taxes	$110,676	72.78	$110,851	65.54	$119,867	63.09	$552,789	60.72	$894,183	62.90
Local	4,705	3.10	6,471	3.83	9,026	4.75	51,907	5.70	72,109	5.07
Tolls	652	0.43	10,262	6.07	5,724	3.01	61,061	6.71	77,699	5.47
Total	116,033	76.31	127,584	75.44	134,617	70.85	665,757	73.13	1,043,991	73.44
Other revenue income:										
Property taxes and assessments	17,970	11.82	24,738	14.63	21,769	11.46	88,627	9.73	153,104	10.77
General fund appropriations	15,297	10.06	12,269	7.25	24,085	12.68	122,149	13.42	173,800	12.22
Miscellaneous taxes and fees	2,756	1.81	4,539	2.68	9,523	5.01	33,878	3.72	50,696	3.57
Total	36,023	23.69	41,546	24.56	55,377	29.15	244,654	26.87	377,600	26.56
TOTAL REVENUE INCOME	152,056	100.00	169,130	100.00	189,994	100.00	910,411	100.00	1,421,591	100.00
Investment income and borrowing	11,552	------	24,889	------	29,678	------	156,727	------	222,846	------
TOTAL INCOME	$163,608	------	$194,019	------	$219,672	------	$1,067,138	------	$1,644,437	------
EXPENDITURES										
Capital outlay:										
On State highways	$82,000	53.42	$97,375	52.18	$90,231	46.50	$416,815	43.57	$686,421	46.04
On local roads and streets	24,601	16.03	26,875	14.40	36,428	18.77	161,081	16.83	248,985	16.70
Total	106,601	69.45	124,250	66.58	126,659	65.27	577,896	60.40	935,406	62.74
Maintenance, administration, operation, etc	44,627	29.08	51,491	27.60	59,031	30.42	320,543	33.51	475,692	31.91
Interest on debt	2,263	1.47	10,866	5.82	8,372	4.31	58,279	6.09	79,780	5.35
Total	46,890	30.55	62,357	33.42	67,403	34.73	378,822	39.60	555,472	37.26
Total expenditures	153,491	100.00	186,607	100.00	194,062	100.00	956,718	100.00	1,490,878	100.00
Debt retirement	8,120	------	24,925	------	19,605	------	88,514	------	141,164	------
TOTAL DISBURSEMENTS	$161,611	------	$211,532	------	$213,667	------	$1,045,232	------	$1,632,042	------
EARNINGS FROM MOTOR-VEHICLE-USER TAXES										
Federal Trust Fund	$47,606	33.08	$57,820	29.73	$71,638	31.78	$322,138	29.67	$499,202	30.26
State motor-vehicle-user taxes	91,029	63.25	119,943	61.67	139,064	61.68	650,720	59.93	1,000,756	60.66
Local motor-vehicle-user taxes	4,640	3.22	6,471	3.33	9,026	4.00	51,907	4.78	72,044	4.37
Tolls	652	0.45	10,262	5.27	5,724	2.54	61,061	5.62	77,699	4.71
TOTAL EARNINGS	$143,927	100.00	$194,496	100.00	$225,452	100.00	$1,085,826	100.00	$1,649,701	100.00

SMSA's studied, road-user income sources provided 73.4 percent of the revenue income, property taxes and assessments and other sources, 26.6 percent. Although the earnings by the travel in these SMSA's amounted to $1,650 million, the amount of road-user taxes, fees, and tolls applied to these SMSA's was only $1,044 million, or 63.2 percent of the amount earned. Earnings of $1.6 billion from the travel in these SMSA's are of the same order of magnitude as: the total receipts for highways, which include $378 million from nonuser sources and $223 million of borrowing; and the total expenditures, which include more than $141 million of debt retirement.

It is not known, of course, whether the totals of all metropolitan areas of the country, if arrayed in this manner, would compare in the same fashion, but it is reasonable to speculate that they might.

A year's expenditures in an SMSA may substantially exceed earnings when highway construction is at a high level (such as in 1960) and a large part of a State's construction program is within the area of the SMSA. This same concentration of construction probably will not continue indefinitely in any one SMSA, and the expenditure-earning relationship would improve. The relationship between expenditures and earnings cited here is valid for only 1 year, but the average for 46 SMSA's may be taken as reasonably indicative of the current trend.

1960 and 1961 data

Because 1960 was the first year for which the complete highway finance data for SMSA's were compiled, it was possible to survey income and expenditures for only that 1 year. Since this analysis was undertaken, however, 1961 data for the 46 SMSA's have become available in sufficient detail to permit a comparison of the State highway department expenditures, excluding those for toll facilities, with 1960 expenditures and to examine any major fluctuations between the 2 years.

State highway department expenditures may increase or decrease from year to year in an SMSA, as shown in table 14. In this table the 1960 and 1961 State highway department expenditures for capital improvements on State and local highways are presented, the increase or decrease is shown in column 3. The 1960 amount of $628 million is a part of the $686 million capital outlay on State highways (tables 9 and 13), but excludes local expenditures on State administered highways and State toll facilities.

Column 4 of table 14 duplicates the total road and street expenditures of $1,491 million given in table 9, then adjusts these amounts for each SMSA by substituting the 1961 State capital outlay for that of 1960. The *adjusted* expenditure is then compared with the 1960 road-user earnings of table 11, and in column 7 of table 14 there is given a new ratio of earnings to expenditures on this basis.

The result of this exercise demonstrates that there is a rather stable relationship between earnings and expenditures for the 2 successive years. In the last two columns of the table the ratios of the 1960 data and the ratios based on the modified 1961 expenditures are presented.

Data for individual SMSA's

The ratio of motor-vehicle user earnings to expenditures for the SMSA's varied considerably: The lowest ratio was 0.34 in the Fargo, N. Dak., SMSA; the highest was 2.95 in the Charleston, W. Va., SMSA. Attempts to rationalize the differences from the standpoint of population, travel, amount of land area, or population density failed to provide a complete answer. Some effect, of course, was the result of the earnings per vehicle-mile of travel that in the Areas cited previously showed the Fargo, N. Dak., SMSA earned 1.283 cents per mile and the Charleston, W. Va., SMSA, 1.534 cents. These differences can be mainly ascribed to a 5- and 6-cent-per-gallon gasoline tax rate in North Dakota—the rate changed from 5 to 6 cents in 1960—compared with a 7-cent-per-gallon gasoline tax in West Virginia. A comparison of capital expenditures in 1960 and 1961, as listed in table 14, shows that in the Fargo SMSA, State expenditures for capital outlay on State and local highways amounted to $11.8 million in 1960 and $5.4 million in 1961, whereas in the Charleston SMSA $0.7 million was expended in 1960 and $3.7 million in 1961.

More striking, perhaps, is the situation in the SMSA's of Chicago and Los Angeles. In the former, capital expenditures were $121 million in 1960 and $61 million in 1961—a reduction in expenditures that, if applied to the 1960 earnings, was sufficient to bring the earnings-expenditure ratio to nearly 1.0. In Los Angeles, capital outlays were $99 million in 1960 and $183 million in 1961. The adjusted expenditures compared with 1960 user earnings retained a ratio of more than 1.0.

The largest extremes between the earnings and expenditures shown in table 11 are in the group having a population of less than 250,000. Two Areas in this group had a ratio of less than 0.50, and five of less than 0.70; whereas only one Area, Albuquerque, in the population groups of more than 250,000 had an earnings-expenditure ratio of less than 0.70. At the other extreme, 8 SMSA's out of 21 in the group having a population of less than 250,000 each had an earnings-expenditure ratio of more than 1.50—38 percent of the group—compared with only 6 SMSA's out of 25 in all of the remaining population groups that had a ratio of more than 1.50.

The more constant relationship in the SMSA's having 250,000 or more population may signify that, at least for 1960—and to the extent that comparisons of the substituted 1961 State highway department expenditures are an indication—there is more stability between the user earnings and expenditures in the larger metropolitan areas even during a period of a high level of highway construction.

Table 14.—Expenditures for capital outlay by State highway departments, 46 SMSA's, 1960 and 1961, and 1960 motor-vehicle road-user earnings [1]

SMSA's by population group	Capital outlay by State highway departments on State and locally administered highways		1961 increase or decrease	Expenditures		1960 road-user earnings	Ratio of user earnings to—	
	1960	1961		Total 1960 (table 9)	Adjusted by substituting 1961 for 1960 State capital outlay		Adjusted expenditures (Expenditures=1.0)	1960 (table 11)
LESS THAN 250,000								
	Thousands	*Thousands*	*Thousands*	*Thousands*	*Thousands*	*Thousands*		
Atlantic City, N.J.	$1,055	$2,363	+$1,308	$6,459	$7,767	$12,445	1.60	1.93
Bay City, Mich.	3,913	2,375	−1,538	6,286	4,748	5,105	1.08	.81
Cedar Rapids, Iowa	1,617	2,377	+760	5,684	6,444	7,323	1.14	1.29
Charleston, S.C.	3,170	6,209	+3,039	4,687	7,726	8,435	1.09	1.80
Eugene, Oreg.	8,102	3,767	−4,335	13,885	9,550	9,329	.98	.67
Fargo, N. Dak.	11,769	5,389	−6,380	17,008	10,628	5,797	.55	.34
Fitchburg-Leominster, Mass.	106	274	+168	1,652	1,820	4,790	2.63	2.90
Fort Wayne, Ind.	2,157	2,181	+24	5,537	5,561	7,566	1.36	1.37
Great Falls, Mont.	2,904	1,815	−1,089	4,871	3,782	3,359	.89	.69
Jackson, Miss.	2,765	2,730	−35	7,196	7,161	7,775	1.09	1.08
Las Vegas, Nev.	3,044	2,850	−194	5,939	5,745	5,577	.97	.94
Lewiston-Auburn, Maine	110	27	−83	1,218	1,135	2,678	2.36	2.20
Lexington, Ky.	1,557	3,649	+2,092	2,703	4,795	5,579	1.16	2.06
Little Rock-North Little Rock, Ark.	17,536	13,352	−4,184	22,171	17,987	11,394	.63	.51
Lynchburg, Va.	1,411	1,173	−238	3,364	3,126	5,317	1.70	1.58
Macon, Ga.	1,627	1,482	−145	2,978	2,833	5,168	1.82	1.74
Madison, Wis.	6,748	15,300	+8,552	15,569	24,121	11,712	.49	.75
Sioux Falls, S. Dak.	7,440	10,111	+2,671	10,008	12,679	4,620	.36	.46
South Bend, Ind.	169	295	+126	3,907	4,033	7,341	1.82	1.88
Springfield, Mo.	3,552	7,431	+3,879	5,612	9,491	6,237	.66	1.11
Waterbury, Conn.	833	4,786	+3,953	6,757	10,710	6,380	.60	.94
Total	$81,585	$89,936	+$8,351	$153,491	$161,842	$143,927	.89	.94
250,000 TO 500,000								
Albuquerque, N. Mex.	$10,076	$8,216	−$1,860	$16,752	$14,892	$11,269	0.76	0.67
Charleston, W. Va.	655	3,708	+3,053	4,234	7,287	12,495	1.71	2.95
Charlotte, N.C.	1,832	1,330	−502	5,581	5,079	9,347	1.84	1.67
Jacksonville, Fla.	11,769	5,761	−6,008	38,310	32,302	26,777	.83	.70
Nashville, Tenn.	16,258	11,558	−4,700	21,641	16,941	16,937	1.00	.78
Omaha, Nebr.	11,697	12,189	+492	24,721	25,213	24,757	.98	1.00
Salt Lake City, Utah	8,270	13,898	+5,628	13,045	18,673	14,094	.75	1.08
Tacoma, Wash.	6,994	9,531	+2,537	13,383	15,920	17,122	1.08	1.28
Tulsa, Okla.	3,771	5,630	+1,859	14,269	16,128	23,770	1.47	1.67
Wichita, Kans.	6,914	7,032	+118	17,899	18,017	15,137	.84	.85
Wilmington, Del.	7,820	6,690	−1,130	16,772	15,642	22,791	1.46	1.36
Total	$86,056	$85,543	−$513	$186,607	$186,094	$194,496	1.05	1.04
500,000 TO 1,000,000								
Birmingham, Ala.	$5,314	$3,964	−$1,350	$14,421	$13,071	$25,084	1.92	1.74
Columbus, Ohio	18,244	14,347	−3,897	28,979	25,082	34,045	1.36	1.17
Denver, Colo.	11,089	11,335	+246	25,018	25,264	41,936	1.66	1.68
Honolulu, Hawaii	7,461	8,015	+554	19,071	19,625	17,815	.91	.93
New Orleans, La.	8,099	7,037	−1,062	37,335	36,273	30,730	.85	.82
Phoenix, Ariz.	2,940	10,757	+7,817	27,440	35,257	34,587	.98	1.26
Providence, R.I.	24,051	23,175	−876	41,798	40,922	41,255	1.01	.99
Total	$77,198	$78,630	+$1,432	$194,062	$195,494	$225,452	1.15	1.16
MORE THAN 1,000,000								
Baltimore, Md.	$26,034	$37,928	+$11,894	$62,209	$74,103	$84,005	1.15	1.35
Buffalo, N.Y.	17,243	14,390	−2,853	56,468	53,615	50,472	.94	.89
Chicago, Ill.	120,734	60,705	−60,029	334,301	274,272	272,158	.99	.81
Houston, Tex.	35,142	44,338	+9,196	71,941	81,137	55,580	.69	.77
Los Angeles, Calif.	99,147	182,551	+83,404	226,185	309,589	386,468	1.25	1.71
Minneapolis-St. Paul, Minn.	42,470	49,586	+7,116	88,109	95,225	65,839	.69	.75
Philadelphia, Pa.	41,921	29,203	−12,718	117,505	104,787	171,304	1.63	1.46
Total	$382,691	$418,701	+$36,010	$956,718	$992,728	$1,085,826	1.09	1.13
TOTAL, All SMSA's	$627,530	$672,810	+$45,280	$1,490,878	$1,536,158	$1,649,701	1.07	1.11

[1] Although local agencies undertake costly facilities also, the impact of interstate construction under State jurisdiction is most certain to affect large and small population areas.

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(*2*) *Highway Statistics 1960,* by Bureau of Public Roads, U.S. Department of Commerce.

(*3*) *Supplementary Report of the Highway Cost Allocation Study,* H. Doc. 124, 89th Cong., 1st sess., 1965.

(*4*) *Improved Transportation for Southeastern Pennsylvania,* Bureau of Municipal Research, and Pennsylvania Economy League, Philadelphia, Pa., May 1960 (Processed).

(*5*) *The Application of Benefit-Cost Ratios to an Expressway System,* by Howard W. Bevis, Proceedings, Highway Research Board, 1956, vol. 35, pp. 63–75.

(*6*) *Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study,* by Arthur A. Carter, Jr., Bureau of Public Roads, U.S. Department of Commerce, May 1962; and *Appendixes to the Original Wisconsin Avenue Study.*

(*7*) *Time and Fuel Consumption for Highway User Benefit Studies,* by Paul J. Claffey, HRB Bulletin 276, Motor Vehicle Time and Fuel Consumption, 1960, pp. 20–34.

(*8*) *Fuel and Time Consumption Rates for Trucks in Freight Service,* by Malcolm F. Kent, HRB Bulletin 276, Motor Vehicle Time and Fuel Consumption, 1960, pp. 1–19.

(*9*) *Motor Transport Fuel Consumption Rates and Travel Time,* by Roy B. Sawhill and Joseph C. Firey, HRB Bulletin 276, Motor Vehicle Time and Fuel Consumption, 1960, pp. 35–68.

(*10*) *Total Receipts for Highways, All Units of Government, 1960–63,* table HF–1, December 1962, news release, Bureau of Public Roads, Department of Commerce, Jan. 13, 1963.

Motor Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in the table on pages 192–193. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of December 31, 1964, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; maximum gross weights for single-unit trucks, truck-tractor semi-trailer combinations, and other combinations; and certain performance standards.

New Publications

Supplementary Report of the Highway Cost Allocation Study

The *Supplementary Report of The Highway Cost Allocation Study,* prepared by the Bureau of Public Roads and transmitted to the Congress on March 24, 1965, by the Secretary of Commerce, John T. Connor, has been published as House Document 124, 89th Congress, 1st session. This report may be obtained from the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402, for $1.00. *The Highway Cost Allocation Study,* Parts I–V (H. Doc. 54, 87th Cong., 1st sess., 1961) also may be purchased from the Superintendent of Documents; the price is 70 cents.

The supplement, as does the basic report, contains information from a study conducted by the Bureau of Public Roads in response to a Congressional directive to provide information for "an equitable distribution of the tax burden among the various classes of persons using the Federal-aid highways or otherwise deriving benefits from such highways." The supplementary report makes use of the final results of the AASHO Road Test, carried out at Ottawa, Ill., under the sponsorship of the American Association of State Highway Officials.

Congress directed that the Federal-aid highway program be studied on the basis of both the costs incurred to serve the different groups of highway users and the benefits they receive from the use of the Federal-aid system. Accordingly, *incremental cost* and *differential benefit* methods of determining cost responsibility were used in the study.

Incremental cost is the traditional method used for determining highway-user charges. Under this method, each element of highway design affected by the size or weight of the vehicles in the traffic stream is broken down into a series of additions, or increments, and the cost of providing each of these additions is charged only to those vehicles whose size and weight require them. In this way, all vehicles will share in the cost of the first or basic increment, but for each succeeding and heavier increment only the vehicles that require the additional design feature will be required to contribute to its cost.

Differential benefit is the method used for assigning cost responsibility to the different groups of users in direct proportion to the vehicular benefits that they receive through their use of the highways. The four kinds of vehicular benefits measured were: (1) reductions in operating costs, (2) reductions in time costs, (3) reductions in accident costs, and (4) reductions in the strains and discomforts of driving. The latter, known as impedance costs, were calculated for passenger-car users only.

U. S. GOVERNMENT PRINTING OFFICE: 1965

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–32 are available upon request addressed to Bureau of Public Roads, Washington, D.C., 20235.*

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ANNUAL REPORTS

Annual Reports of the Bureau of Public Roads:
1960, 35 cents. 1963, 35 cents. 1964, 35 cents. (Other years are now out of print.)

REPORTS TO CONGRESS

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:
Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.
Supplementary Report, House Document No. 124 (1965). $1.00.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42 (1965). 20 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways, 1937–61, $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

Calibrating and Testing a Gravity Model With a Small Computer (1964). $2.50.

Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

Design of Roadside Drainage Channels (1965). 40 cents.

Federal Laws, Regulations, and Other Material Relating to Highways (1960). $1.00.

Financing of Highways by Counties and Local Rural Governments: 1942–51 (1955). 75 cents.

Highway Bond Financing . . . An Analysis, 1950–1962. 35 cents.

Highway Finance 1921–1962 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

Highway Planning Map Manual (1963). $1.00.

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

PUBLICATIONS—Continued

Highway Research and Development Studies Using Federal-Aid Research and Planning Funds (1964). $1.00.

Highway Statistics (published annually since 1945): 1956, $1.00. 1957, $1.25. 1958, $1.00. 1959, $1.00. 1960, $1.25. 1961, $1.00. 1962, $1.00. 1963, $1.00.

Highway Statistics, Summary to 1955. $1.00.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Highways and Economic and Social Changes (1964). $1.25.

Hydraulics of Bridge Waterways (1960). 40 cents.

Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). 40 cents. Appendix, 70 cents.

Interstate System Route Log and Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal-Aid Construction, 2d ed. (1965). $1.75.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.

Part V—Traffic Controls for Highway Construction and Maintenance Operations (1963). 25 cents.

Opportunities for Young Engineers in the Bureau of Public Roads (1964). 15 cents.

Peak Rates of Runoff From Small Watersheds (1961). 30 cents.

Reinforced Concrete Pipe Culverts—Criteria for Structural Design and Installation (1963). 30 cents.

Road-User and Property Taxes on Selected Motor Vehicles, 1964. 45 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways (1958): a reference guide outline. 75 cents.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-61 (1962–1964). $2.25.

Standard Plans for Highway Bridges (1962):
Vol. I—Concrete Superstructures. $1.00.
Vol. II—Structural Steel Superstructures. $1.00.
Vol. III—Timber Bridges. $1.00.
Vol. IV—Typical Continuous Bridges. $1.00.
Vol. V—Typical Pedestrian Bridges. $1.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Aerial Surveys in Highway Engineering (1960). 40 cents.

Traffic Assignment Manual (1964). $1.50.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Transition Curves for Highways (1940). $1.75.

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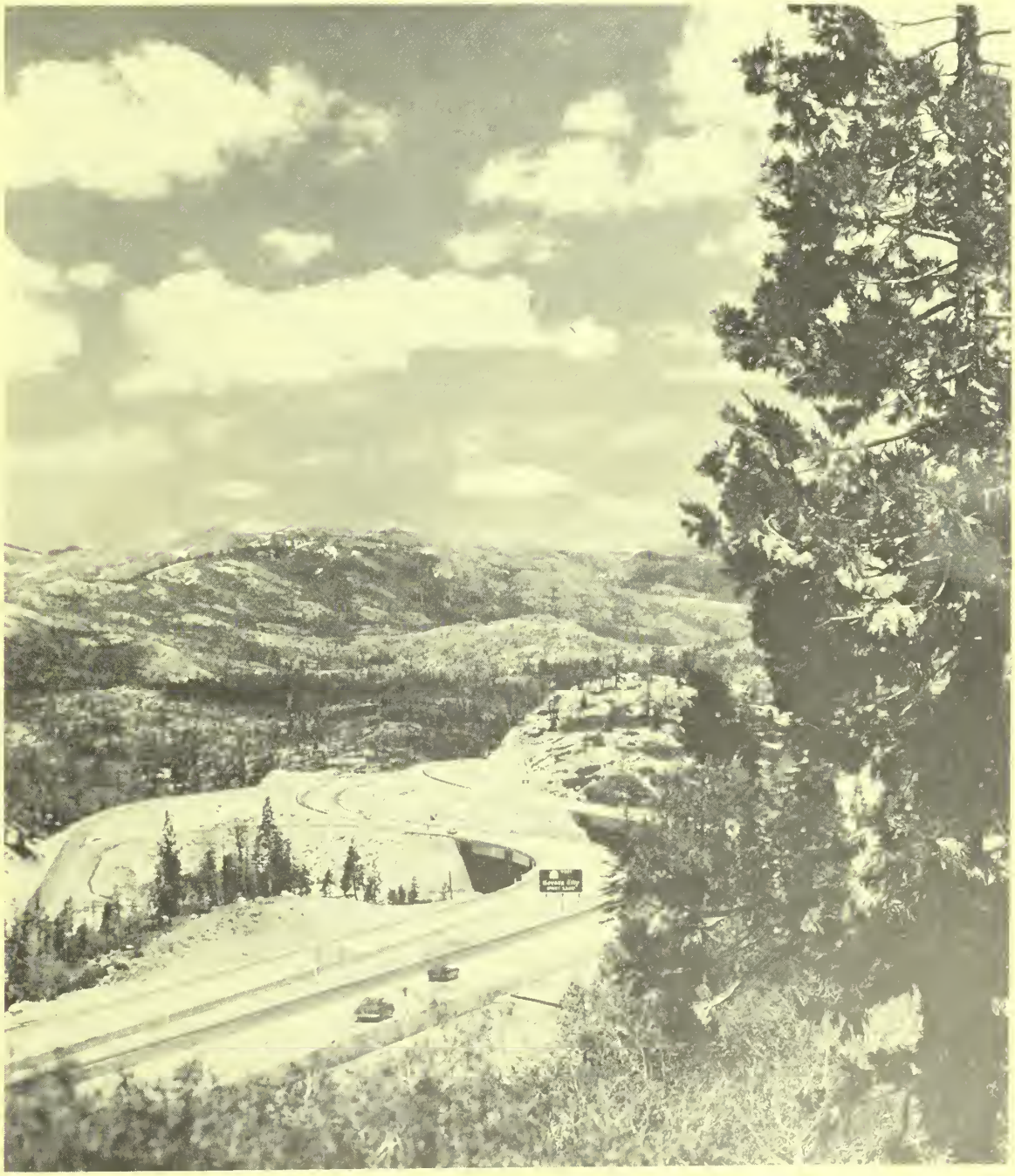

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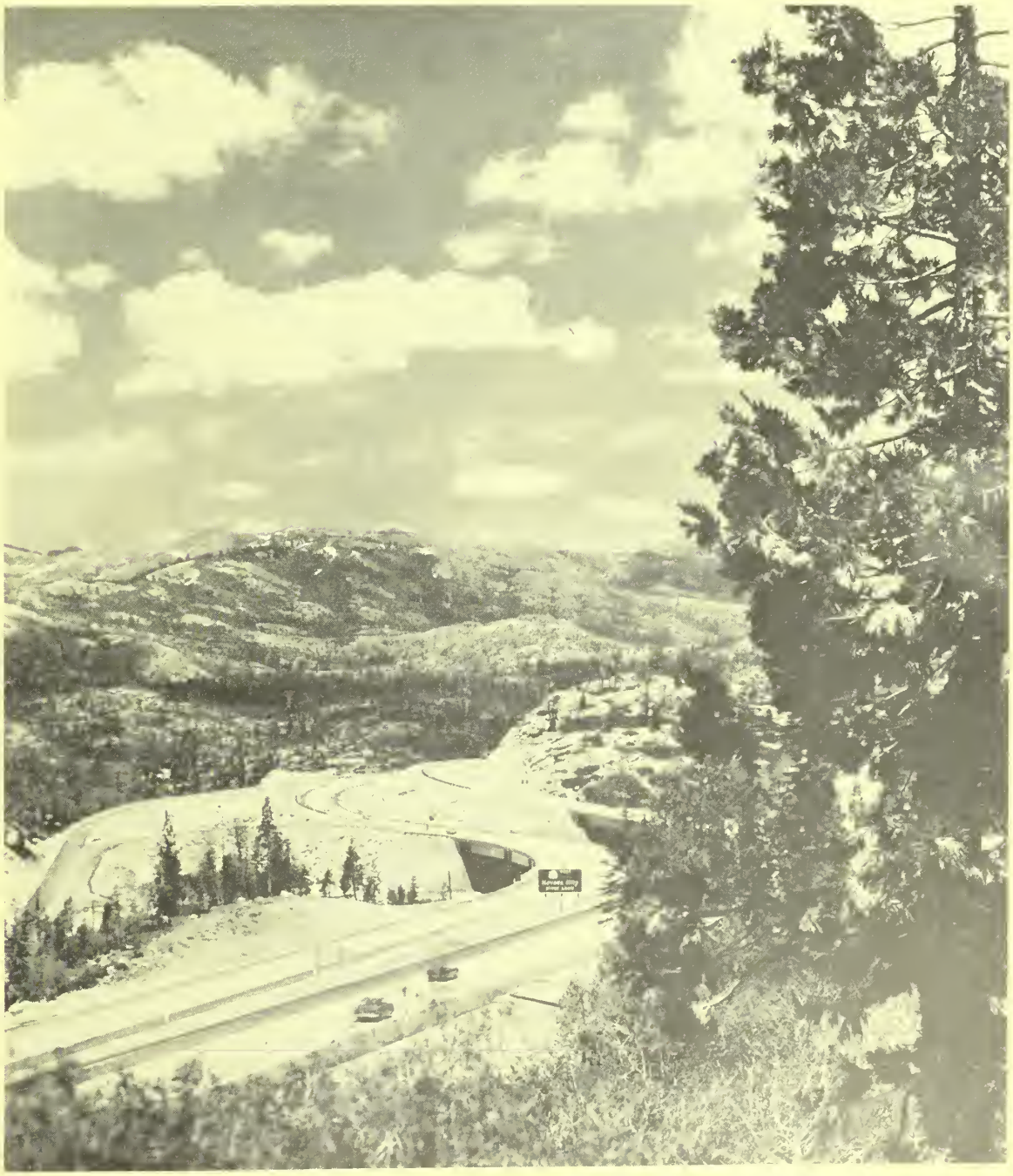

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IN THIS ISSUE

Abrasion Resistance of Bridge Paints for Use in Alaska—Field and Laboratory Tests Evaluated, *by Bernard Chaiken* 201

Voids, Permeability, and Film Thickness Related to Asphalt Hardening, *by J. F. Goode and L. A. Lufsey* 211

New Publications 221

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 6, 1961.

Contents of this publication may be reprinted. Mention of source is requested.

Abrasion Resistance of Bridge Paints for Use in Alaska— Field and Laboratory Tests Evaluated

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

Reported by[1] BERNARD CHAIKEN, Principal Research Chemist, Materials Division

Findings from a study of the resistance of paint systems to abrasion in the Alaska Copper River Delta area are reported in this article. The study was instituted because of the rapid and premature failure of a conventional highway paint system applied to old railroad bridges modified for highway use as part of the Copper River Highway-Edgerton Cutoff. Tests were made to determine the most suitable paint system for this area where it must resist abrasion from extremely large amounts of aeolian sand, silt, and ice crystals carried by winds having high velocities, as much as 90 m.p.h.

A field study and accelerated laboratory abrasion test were made to evaluate nine different paint systems. The superiority of a catalyzed polysulfide paint system was demonstrated by results of the field study and confirmed by the laboratory tests. The other paint systems evaluated were coal tar, drying oil, alkyd, vinyl, epoxy ester, zinc-rich inorganic, neoprene, and a conventional red lead primer and aluminum varnish system.

Laboratory confirmation was needed to assess the validity of the field findings as the thicknesses for the applied paint systems varied considerably in the field study. The laboratory tests confirmed that the polysulfide coating was inherently more resistant to abrasion than the other paints when comparable film thicknesses were tested. The author recommends that a catalyzed liquid polysulfide rubber coating replace the ordinary red lead primer and aluminum varnish paint system for maintenance of steel bridges exposed to the extreme abrasive forces prevalent in the Copper River Delta of Alaska. He recommends that the dry coating thicknesses be limited to 12 to 15 mils and suggests that incorporation of a rust-inhibitive primer would improve the resistance of the paint system to corrosion.

Introduction

DURING the period 1953 to 1957, several abandoned railroad bridges in the Copper River Delta area of Alaska, which had been built and operated for some years by the Copper River and Northwestern Railroad, were modified for highway use as part of the Copper River Highway-Edgerton Cutoff, FAS Route 851, State Route 10. Both the completed and proposed portions of the new highway will eventually make use of the abandoned roadbed of the old railroad between Cordova and Chitina. The completed portion of this highway is shown as a solid line in figure 1.

In 1957, all new structural steel and guardrails on several of the modified bridges were shop-primed and field-coated with a red lead oil-alkyd paint conforming to Federal Specification TT–P–86a, Type II. A two-package aluminum paint, conforming to Federal Specifications TT–A–468a and TT–V–81b, was used as the finish coat. This paint system, in common use in highway work, failed completely within 6 months in places where erosion by windblown sand and ice particles was extreme. Several examples of extremely rapid paint failures and subsequent corrosion are shown in figures 2 through 7. It was obvious that a much more abrasion-resistant paint system was needed to protect these bridges of the Copper River Delta, as well as bridges in other areas where wind-related abrasion is extreme.

The lower Copper River Valley between the town of Chitina and the Gulf of Alaska seems to be the only low-elevation break in the Chugach Mountains through which large masses of air pass freely from the interior high-pressure areas to the low-pressure systems common in the Gulf of Alaska. The resultant high-velocity north winds, at times as high as 90 m.p.h. or more, sweep great volumes of sand and silt into the Copper River Delta where several old railroad bridges have been converted to highway use. The location of these bridges is shown in figure 1. Although no precise wind data are available from the lower Copper River Valley, it is known that the old Copper River and Northwestern Railroad at one time had an anemometer destroyed by winds of approximately 90 m.p.h. There are also old reports of loaded copper ore cars having been overturned by the extreme force of the wind. Maximum winds of 70–80 m.p.h. have been recorded at the Cordova airport, near the coast, some 14 miles west of the Mile 27 Bridge. However, it is known that stronger winds are common at the bridge sites in the Delta.

The windborne sand and silt comes from the interior glaciers as well as numerous sandbars located in the Delta. During the winter months the quantity of aeolian sand and silt is somewhat reduced but the windborne ice particles then become the main agent of erosion. In addition to persistent northerly winds, the Delta is subject to 98 inches of precipitation per year as measured at the Cordova airport. The general area is subject to a low annual mean temperature of 28° F. (21° F. in December to 53° F. in July).

After the rapid failure of the standard paint system applied to the newly erected steel in 1957, a careful inspection of the older parts of the structures was made to ascertain the condition of the paint that had been used by the railroad some years before. This older paint seemed to be in fairly good condition, as can be seen in figure 8. As a result, an effort was made to determine the nature of the paint used by the railroad and the history of its painting operations.

The railroad, completed about 1910, was built to serve the Kennecott Copper Mines in the Wrangell Mountains, some 160 miles from the seaport town of Cordova. The railroad

[1] The work reported here was performed in cooperation with the Planning and Research Section, Alaska Department of Highways, the Materials Section, Anchorage Division of the U.S. Bureau of Public Roads Regional Office, and the Public Roads research laboratories in Arlington, Va. Work performed by the State was financed with Federal-Aid funds.

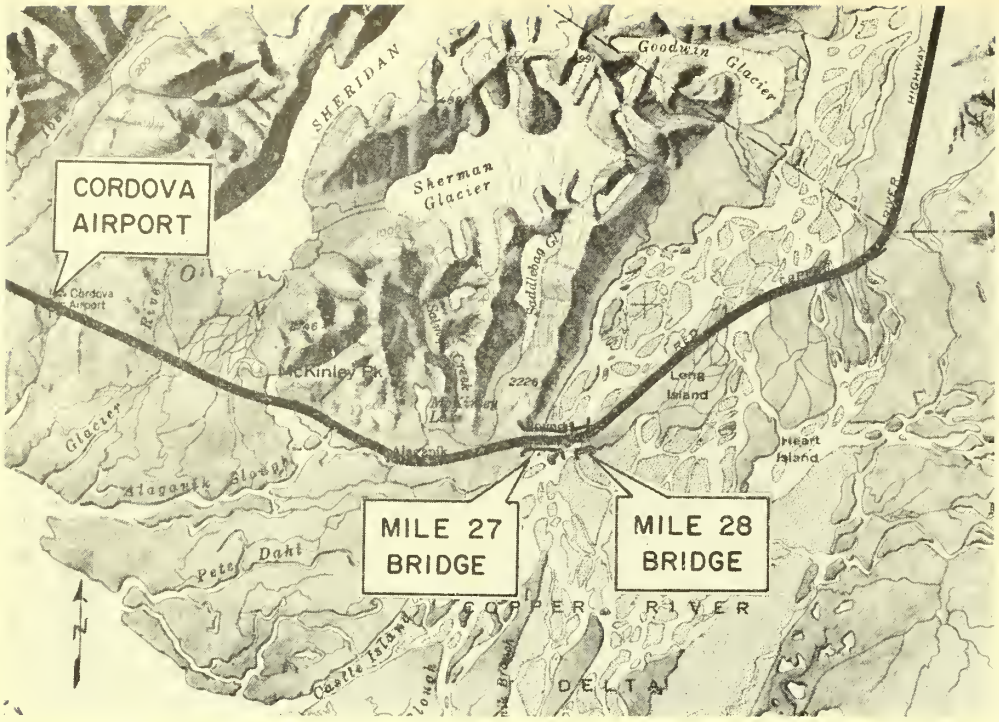

Figure 1.—Completed section of the Copper River Highway, Alaska, is shown by the solid line. Sites for the paint tests are also indicated, Mile 27 and Mile 28 Bridges at the Copper River Delta.

Figure 2.—Paint erosion on guardrail of east abutment of Mile 27 Bridge.

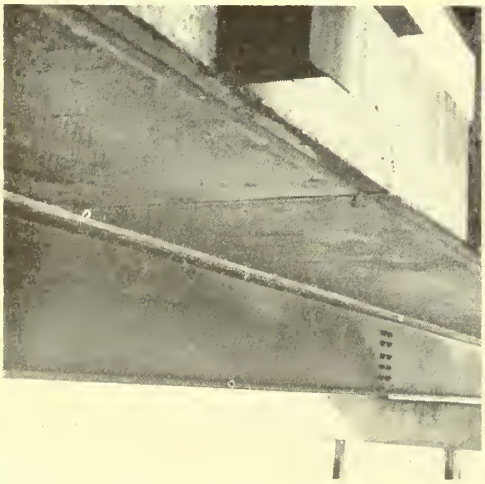

Figure 3.—Severe erosion of paint on steel and even pitting of the concrete on north side of east end of Mile 27 Bridge.

was operated through 1937, at which time the Kennecott mining operations were terminated. Records and field examination showed that the first paint system used on several of the railroad bridges, including those at Mile 27 and Mile 28, was a red lead shop primer, a field primer of *Marine Tockolith*, and a final coat of *RIW Waterproof Black*. Subsequent correspondence with the original paint supplier for these projects indicated that the final field coat was undoubtedly their *RIW No. 49 Black*, which was commonly used in that period, over an intermediate coat of their *RIW Marine Tockolith* (now called *RIW Tockolith*). It was further determined that the first repainting of the railroad bridges was done in 1917, some 7 years after the initial construction. In repainting, the metal was scraped and wire-brushed by hand, then brush-coated with a red lead and linseed oil primer, and brush topcoated with *Metalastic Black*. A routine spot repainting program, in which the 1917 paint system was employed, was reinstituted in 1921 and continued until the railroad was abandoned in 1937, according to a letter from Martin Fredeen, who was in charge of maintenance on the Copper River and Northwestern Railroad.

In 1958, much of this maintenance paint continued to provide effective protection to the unmodified structural steel after more than 25 years of exposure to the harsh climatic conditions described. Efforts in 1958 to obtain the *Metalastic* topcoat paint from the manufacturer were unsuccessful as its manufacture had been discontinued. Even the generic composition of this paint was unavailable at that time. However, subsequent to the completion of the study described in this article, private communications with R. T. Gabbert of the Sherwin-Williams Co. provided information on the probable composition of the Metalastic topcoat used in maintenance painting of the railroad bridges. His review of the company's old literature suggested that the Metalastic black topcoat paint used in the 1917–21 period may have been their Metalastic Black No. 2, a graphite drying oil paint having a probable composition of 32.8 percent pigment and 67.2 percent vehicle, which probably was a linseed or linseed tung oil blend. The pigment probably was composed of materials and percentages, as listed: Graphite, 54.4; fibrous talc, 29.3; amorphous silica, 9.4; carbon (channel) black, 4.4; litharage, 2.5.

Conclusions and Recommendations

From the paint systems studied and the results obtained, it is concluded that thick films of a catalyzed liquid polysulfide rubber coating (like System 8) should offer the best protection for steel bridges exposed to the extreme abrasive forces prevalent in the Copper River Delta of Alaska and that this system should replace the ordinary red lead primer and aluminum varnish system in standard use in that area. A word of caution and perhaps some modification of System 8 might be in order to assure good practical results. In field exposures D and E some sagging of the polysulfide system occurred perhaps as a consequence of too thick a film. It is therefore recommeded that the paint thickness be restricted to a dry thickness range of 12 to 15 mils to prevent the occurrence of sagging. It is of interest that the manufacturer of this coating recommended a 40-mil thickness. In a paint system similar to No. 8, inclusion of a rust-inhibitive primer would probably be very beneficial in increasing the paint's resistance to corrosion. This conclusion was reached because the panels used in the field study were sandblasted before being painted. Such a surface pretreatment would not always be practical—hand cleaning is more often used—and the danger of metal corrosion would be greater unless an adequate primer were used.

Purpose and Scope of Investigation

Because of the premature failures of the standard paint system applied in 1957, and the unavailability of the effective maintenance paint used previously by the railroad, an outdoor exposure study was planned in 1958 and initiated in 1959 by the Materials Section, Anchorage Division, of the Bureau of Public Roads Regional Office in Alaska. This exposure study was later taken over—the Alaska Department of Highways assumed primary highway responsibility in July 1960—and completed by the Planning and Research Section of the Alaska Department of Highways.

A specification for a paint system providing adequate resistance to the natural abrasive forces encountered in the Copper River Delta was established as the ultimate goal for the field study. To arrive at this specification, different paint systems were exposed and their performance under outdoor exposure was evaluated. Many paint manufacturers, suppliers, and other authorities

were consulted concerning the most suitable paint systems to be studied. Nine different paint systems were finally selected for evaluation, including as a control the standard system then in use in Alaska.

The paint systems selected for evaluation are identified, as follows: System No. 1, coal tar; System No. 2, drying oil; System No. 3, alkyd; System No. 4, vinyl; System No. 5, epoxy ester; System No. 6, zinc-rich inorganic; System No. 7, neoprene; System No. 8, polysulfide rubber; and System No. 9, aluminum varnish over red lead primer. Some explanations of a few of the paint systems are given here.

System No. 2 was presumably representative of the original *Tockolith* paint system used by the railroad during the original construction of several of the bridges in 1907–09 and was one that reportedly gave fairly good service prior to the use, starting in 1917, of the *Metalastic* maintenance paints.

System No. 3 was supposed to have been a phenolic system and was so represented by the paint supplier. However, subsequent laboratory analysis of this system, after the field study was well underway, revealed that it was alkyd based rather than phenolic.

System No. 9 was supposed to be the same as that used on the new steel and was intended to be the control system included for comparison. However, it was somewhat different than had been anticipated. The primer did not strictly conform to Federal Specification TT–P–86a, Type II, as it contained zinc oxide and zinc chromate along with red lead, iron oxide, and siliceous matter. In addition, the aluminum finish paint was ready-mixed rather than a two-package paint.

The investigation originally had been limited to an outdoor exposure study on bridges of the Copper River Highway, where severe paint deterioration had been noted. The paint systems were to be applied at normal coating thicknesses or as recommended by the paint supplier or producer. After the field investigation was well underway, it seemed worthwhile to make a complete chemical analysis of the paints used and to make laboratory abrasion tests for evaluation of the effects of film thickness differences. Accordingly, samples of the paint were sent to the Arlington laboratories of the U.S. Bureau of Public Roads, Materials Division for these purposes.

Field Exposures

Two bridges of the Copper River Highway located at the Copper River Delta were selected for the field exposure tests, which were started in August 1959. These were the Mile 27 Bridge and Mile 28 Bridge, 14 miles east of the Cordova airport. Five types of paint exposure conditions were arranged. They will be referred to as exposures A, B, C, D, and E. They were selected to provide a range from severe to negligible erosion, as previously noted on the structural steel. In exposures A, B, and C, painted panels were strapped to different parts of the bridges, and in exposures D and E, actual parts of the steel structures were coated with the paint systems being investigated.

Exposures A, B, and C

For exposures A, B, and C, 54 steel panels, 3/16- by 12- by 18-inches in size, were prepared. All edges were ground smooth, and the test surfaces were sandblasted to a uniform bright gray metallic color to remove millscale and rust. Immediately before its being painted, each panel was cleaned with inhibited 1,1,1-trichloroethane solvent by cloth, and the prime coat was brushed on; the rest of the paint system was then applied, also by brush. Only one side of each panel was so coated and each of the nine paint systems was applied to six panels. The coating film thickness and drying time between coats used were those recommended by each supplier or manufacturer. Application data concerning the test panels for exposures A, B, and C are shown in table 1. Generally, the drying time between coats was overnight; the exceptions for special coatings and washcoats are listed in table 1.

In exposure A, one panel of each paint system was clamped to a mounting board attached to the vertical guardrail members on the north side of the eastern abutment of Mile 27 Bridge. Consequently, each panel was in a vertical position facing north and windward. This exposure was considered to be the most severe with respect to windborne abrasive elements. The arrangement of panels in this exposure is shown in figures 9 and 10. The arrangement shown in figure 10 seemed to have some significance, which will be discussed later, because the wind direction was from 15° below and 45° to the left of being perpendicular to the plane of the panels. Consequently, abrasive action was greatest at

Figure 4.—Severe paint erosion on guardrail of east end of Mile 27 Bridge.

Figure 5.—Severe erosion on windward side of I-beam piles on Mile 27 Bridge.

Figure 6.—Close-up view of erosion shown in figure 5.

Figure 7.—Paint erosion on end post of Mile 28 Bridge.

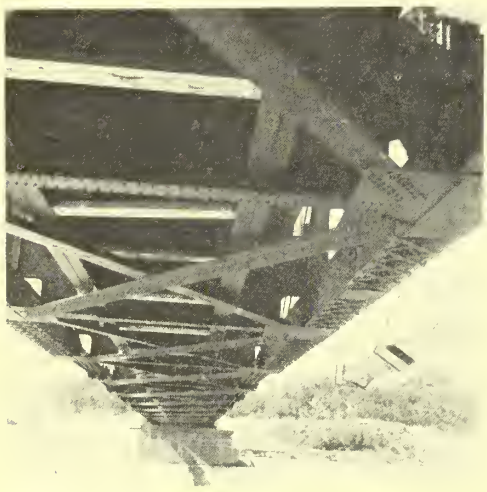

Figure 8.—Sound condition of Metalastic Black maintenance paint used by railroad on underside of Mile 28 Bridge.

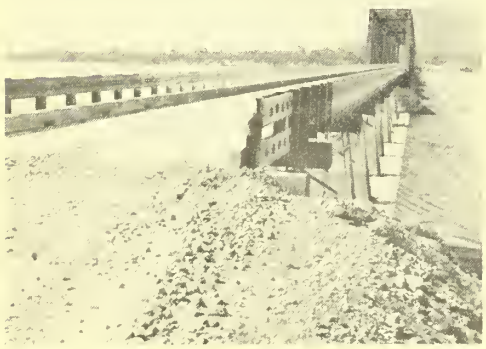

Figure 9.—Test mounting boards for exposure A near end of right guardrail at east abutment of Mile 27 Bridge.

Table 1.—Data on application of Paint Systems to panels for exposures A, B, and C [1]

Paint System	Individual coatings				Dry film thickness [2]	
	Name	Color	Coats applied	Minimum drying time per coat	Individual coating [3]	Total for Paint Systems
			Number	*Hours*	*Mils*	*Mils*
No. 1, Coal tar:						
1-1 Washcoat	Vinyl butyral [4]	Olive, clear	1	1	0.2–0.5	6–7
1-2 Primer	Coal tar cutback	Black	1	24	3.5	
1-3 Finish	Coal tar emulsion	do	1		2–3	
No. 2, Drying oil:						
2-1 Primer	White lead and zinc oxide in oil.	Gray	1	48	0.8–1.0	3–4
2-2 Finish	Carbon black and red lead in oil.	Black	1	24	2.2–3.0	
No. 3, Alkyd:						
3-1 Primer	Red lead alkyd	Brown	1	24	0.5–0.8	1.5
3-2 Finish	do	Orange	1	24	0.8–1.0	
No. 4, Vinyl:						
[5] 4-1 Washcoat	Vinyl butyral [4]	Olive, clear	1	1	0.2–0.5	3.5–4.0
[6] 4-2 Finish	Titanium dioxide vinyl	White	3	Several	3.2–3.8	
No. 5, Epoxy ester:						
5-1 Primer	Red lead epoxy ester	Brown	1	24	1.0–1.5	3.0
5-2 Finish	Titanium dioxide epoxy ester	White	1	24	1.5–2.0	
No. 6, Zinc-rich inorganic:						
6-1 Finish	Inorganic zinc silicate [7]	Gray	2	1	3–4	3–4
6-2 Cure	Phosphoric acid solution	Colorless	2	Several	0	
No. 7, Neoprene:						
7-1 Primer	Neoprene solution	Amber, clear	1	<24	0.2–0.5	4
7-2 Finish	Carbon black and neoprene	Black	1	24	3.5–3.8	
No. 8, Polysulfide:						
8-1 Primer	Chlorinated rubber	Colorless	1	<24	0.5	15–20
8-2 Finish	Catalyzed polysulfide [8]	Black	1	48	15–20	
No. 9, Aluminum varnish:						
9-1 Primer	Red lead alkyd	Brown	1	24	0.5–0.8	2
9-2 Finish	Aluminum oleoresinous varnish. [9]	Aluminum	1	24	1.2–1.5	

[1] 3/16- by 12- by 18-inch steel panels. Panels sandblasted on one side to bare metal to remove millscale; cleaned with inhibited 1, 1, 1-trichloroethane by cloth; and coating brushed on.
[2] Determined magnetically by Elcometer gage.
[3] Average or range for the six panels.
[4] One part catalyst to six parts resin by volume.
[5] Same as washcoat 1-1.
[6] Paint appeared rubbery and did not brush well.
[7] 23 pounds zinc dust to 3/4 gallon of liquid component.
[8] One part accelerator to 10 parts base resin by weight.
[9] Ready-mixed.

the left and bottom where System 6 was located, and least at the top right, where System 5 was located.

In exposure B, four replicate panels of each paint system were clamped to mounting boards attached to the east end of the northern lower chords of Mile 28 Bridge. The panel exposures were vertical, north, and 45° from windward; the same as in exposure A. However, exposure B was considered to be less intense than that of exposure A. The arrangement of these panels is shown in figure 11. All four panels of each system were grouped together, Systems 1, 2, 3, and so on, arranged in increasing order from east (left) to west (right).

Exposure C was the least severe of the three panel exposures, as it was in a somewhat sheltered area. This exposure was used to obtain a relative measure of coating performance under minimal wind conditions. In this exposure, one panel of each paint system was clamped to a mounting board attached to the opposite, and therefore protected (south) side of the same chord on Mile 28 Bridge as for exposure B. Panel positions were thus vertical and facing south. The arrangement of the panels for Systems 1 through 9 is shown in figure 12.

Exposures D and E (paint on bridge surfaces)

In addition to the three types of panel exposures, each of the paint systems was applied directly in exposures D and E to parts of the bridge surface of the Mile 28 Bridge to simulate actual painting practices. The bridge surfaces were either wire-brushed or left unprepared as necessitated by the observed condition of the existing paint on the steel. An attempt was made to obtain the same film thickness with each applied coating as had been obtained in the test panel series. In both of these exposures, the paint was applied to the vertical faces (inboard web surfaces) of the top eastern end posts: System 1 paint was applied at the top and the other systems were applied in order downward to System 9 at the bottom.

At exposure D, which faced north, most of the original field paint had been eroded away; consequently, this area was wire-brushed to remove rust and loose scale before the test paints were applied. Exposure here was vertical and north and therefore subject to considerable erosion. At exposure E, the original paint on the bridge surface was in good condition and therefore no special precleaning was done. These test paints were

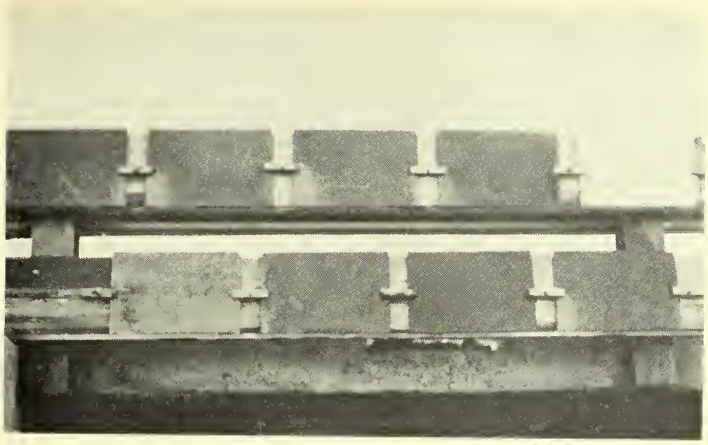

Figure 10.—Close-up view of panel arrangement at exposure A (top row, Systems 1 through 5; bottom row, Systems 6 through 9).

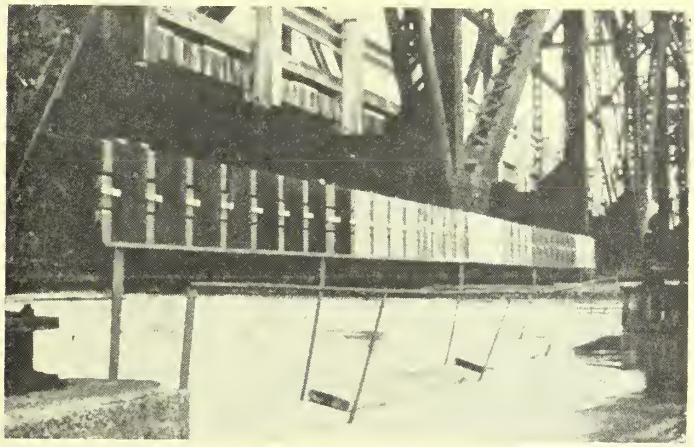

Figure 11.—Panel arrangement at exposure B on east end of northern lower chords of Mile 28 Bridge.

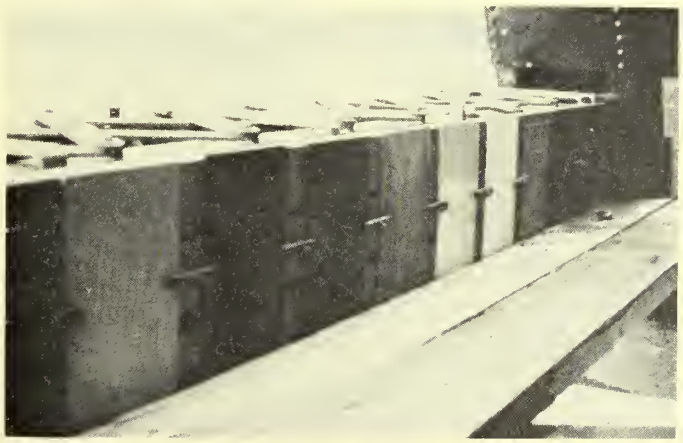

Figure 12.—Exposure C. Panels attached to same chord as exposure B, but on opposite side, facing south.

Figure 13.—Exposure D at Mile 28 Bridge faced north and subject to erosive forces.

Figure 14.—Exposure E at Mile 28 Bridge faced south and protected.

also applied in a vertical plane, but facing south so that effective wind forces were not as severe as in exposure D. The arrangement of painted surfaces in these two exposures is shown in figures 13 and 14.

Results and Evaluation of Field Exposures

During the exposures, the paint systems were periodically rated visually for factors of general appearance, erosion, rusting, gloss, chalking, checking, cracking, flaking, scaling, peeling, dirt, mildew, fading, darkening, and yellowing. In addition, blistering and sagging were noted when evident. These ratings were made on a 10 to 0 scale; 10 is perfect and 0 represents complete failure, in accordance with the standard procedures and pictorial standards provided for these factors by the American Society for Testing Materials (*1*).[2] Because only the trends in general appearance, erosion, and rusting seemed to be significant, the observed ratings for the other factors listed have not been included in this article so that the volume of reported data could be kept at a minimum. The test panels in exposures A, B, and C also were periodically measured for film thickness of the remaining coating by a magnetic dry-film thickness gage to obtain information that could be used to evaluate more fully the erosion resistant properties of the paint system. Such thickness measurements were not made on the paints applied directly to the structure in exposures D and E.

[2] References indicated by italic numbers in parentheses are listed on page 113.

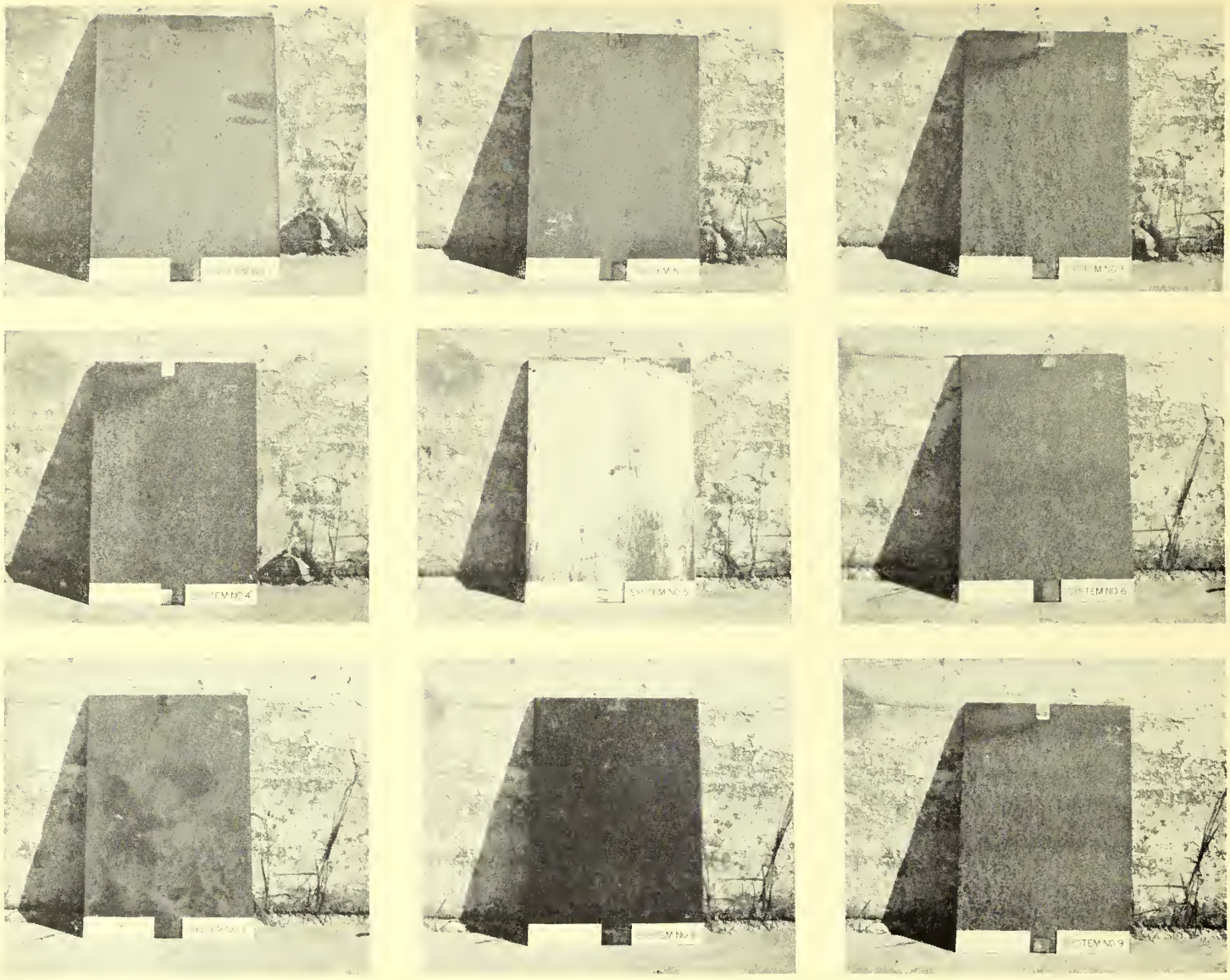

Figure 15.—Condition of Paint System panels after termination of exposure A.

The condition of the paint systems after 38 months at exposure A is illustrated on the nine panels photographed for figure 15. As had been expected, exposure A was confirmed as being the most severe of the five exposure conditions. Although the full extent of rusting is not too clearly shown by the black-and-white photographs, a field examination of these nine panels showed that, except for paint Systems 5 and 8, all systems tested failed badly. However, System 5 had substantial rusting, but paint System 8 was still in very excellent condition. The comparative performance of each of the paint systems is shown more clearly in figure 10, which was photographed during the exposure, and the superiority of Systems 5 and 8 are more clearly delineated.

The harsh effects of exposure A as compared to exposures B and C are demonstrated vividly in figure 16; the left, middle, and right panels in each block are from exposures A, B, and C, respectively. The panels on the left, from exposure A, were each time in the poorest condition.

The final ASTM numerical ratings for all five exposure conditions are shown in table 2 in terms of general appearance, erosion, and rusting. These results reflect quantitatively the condition of each paint system after either 23 or 38 months of exposure, depending upon when the particular exposure was terminated.

The tabulated results show that in exposure A, the most severe, only System 8, polysulfide rubber, provided complete protection to the underlying metal after the full 38 months of exposure. The general appearance and rust resistance of this system was considered to be excellent, and it received a rating of 10. Its performance in the other four, but less severe, exposures was equally good, except for some sagging noted in exposures D and E that perhaps was caused by the application of too thick a film of paint. System 5, epoxy ester, was the only coating other than System 8 that still provided some significant protection to the metal in exposure A after 23 months. However, performance of System 5 was considerably poorer than that of System 8 (fig. 15 and table 2). In the other exposures, B through E, the epoxy ester system appeared to give fair performance, as shown in table 2, but it was outranked or equaled by Systems 1 and 2, as well as System 8. Personnel conducting the field study were of the opinion that System 5 had unknowingly been placed in a favored position in exposure A because of the wind direction at that exposure, as described earlier. This might also be the explanation for the relative difference in this system's performance at site A as compared to the other exposure sites.

To arrive at some overall quantitative evaluation of the paint systems in all five exposures, the numerical ratings for the important factors, such as general appearance, erosion, and rusting at the age of 23 months, were added together for all five exposures. The results are shown in table 3. It is obvious from the cumulative total rating, that System 8 demonstrated overall superiority, having a cumulative total rating of 141 out of a possible maximum rating of 150. Systems 2 and 5 were next; they had ratings of 117 and 116, respectively. The presumably

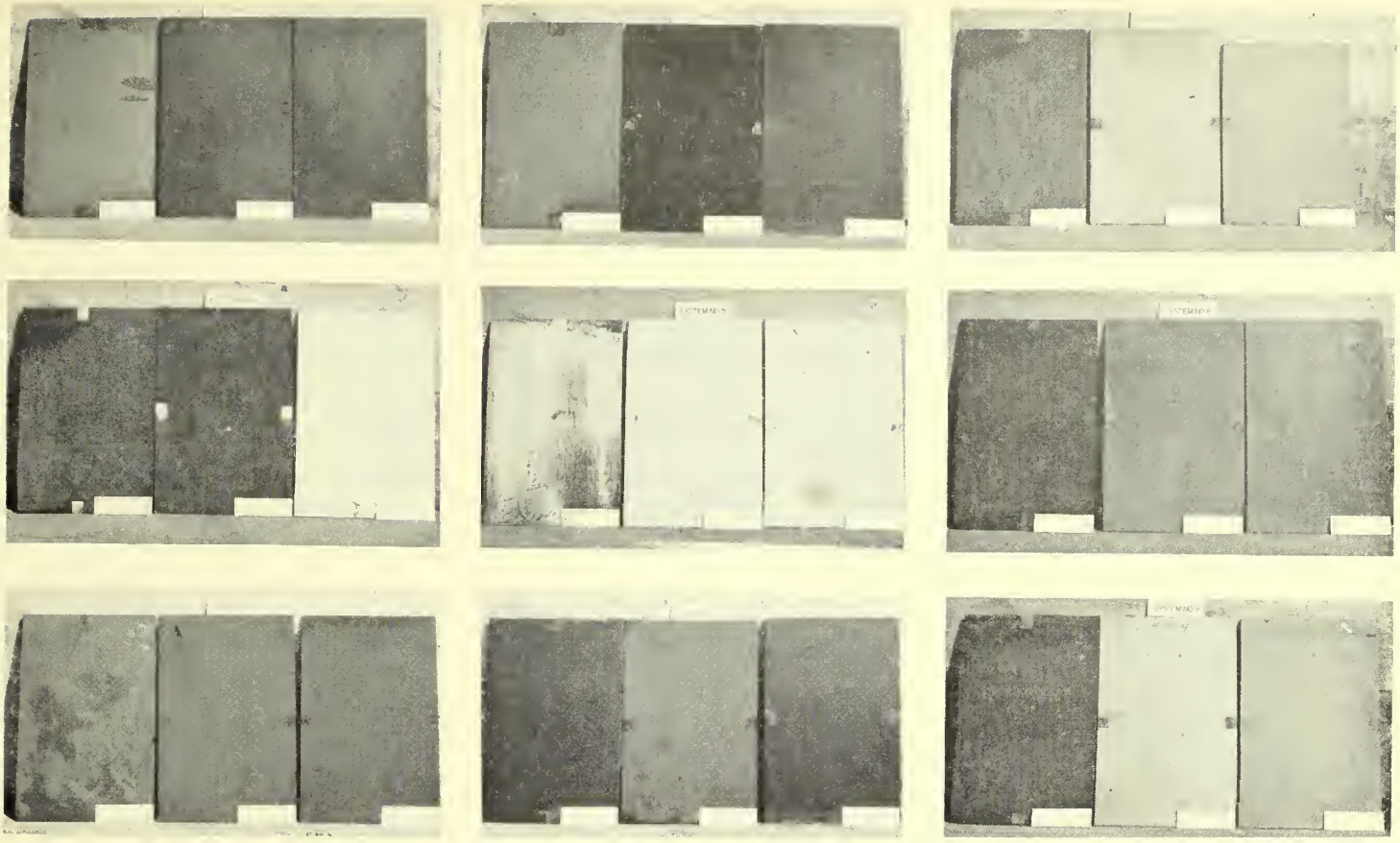

Figure 16.—Comparative condition of Paint Systems after exposures A, B, and C.

standard system, No. 9, was inferior to many of the paint systems studied, and the vinyl and zinc-rich paint systems (4 and 6) were among the most inferior.

Information on the important factor of rusting was used to explore the trends on rusting under the most severe test of exposure A. The change in the amount of rusting in relation to exposure time is shown in figure 17. The results shown here confirmed the impressions formed from other data that System 8 was superior to the other paint systems and indicated that it should have an expected life far in excess of the 3-year test of severe exposure.

Although System 5 had the next best performance, on the basis of test results, it could not be expected to give maintenance-free service for more than 2 to 3 years. This conclusion is based on a rule-of-thumb principle in maintenance painting of steel that when the numerical value of resistance to rusting diminishes to 6, repainting becomes necessary. The resistance to rusting of System 5 diminished to 6 after 23 months of exposure.

Effect of original film thickness

A serious question might be raised regarding the entire exposure study concerning the extent to which the original film thickness of each coating contributed to ultimate performance. It could be argued that, by increasing the original film thickness of some

Table 2.—Panel ratings for general appearance, erosion, and rusting [1]

Condition rated	Paint System—								
	1	2	3	4	5	6	7	8	9
EXPOSURE A, 23 MONTHS [2]									
General appearance	0	0	0	0	4	0	2	10	0
Erosion	0	0	0	0	6	0	3	9	0
Rusting	0	1	0	1	6	2	2	10	2
EXPOSURE B, 38 MONTHS									
General appearance	7	9	4	0	8	1	8	9	5
Erosion	9	10	8	0	7	1	8	10	6
Rusting	10	[3] 10	8	0	8	2	7	10	[3] 8
EXPOSURE C, 38 MONTHS									
General appearance	8	9	6	6	8	1	7	8	3
Erosion	10	10	9	9	10	1	5	10	4
Rusting	10	10	[3] 8	[3] 8	7	2	6	10	[3] 4
EXPOSURE D, 23 MONTHS									
General appearance	10	10	10	1	8	0	9	9	9
Erosion	8	8	7	1	8	0	8	8	6
Rusting	10	10	10	6	[3] 8	4	10	[4] 10	[3] 8
EXPOSURE E, 23 MONTHS									
General appearance	8	10	9	2	8	4	10	8	8
Erosion	6	10	8	1	10	([5])	10	10	8
Rusting	[3] 10	[3] 10	10	10	10	7	10	[4] 10	10

[1] ASTM method of rating where 10=perfect and 0=complete failure.
[2] Ratings after 38 months of exposure A was: for System 5, general appearance 2, erosion 1, and rusting 3; for System 7, general appearance 0, and erosion 0; for System 8, general appearance 10, erosion 9, and rusting 10.
[3] Some blistering noted.
[4] Some sagging noted.
[5] Erosion rating not made.

of the apparently inferior paints, better performance could have been attained. This is a reasonable and legitimate point and is discussed in more detail in relation to the laboratory studies. However, some consideration is given here to the field exposure data collected on film thickness.

The data obtained on the film thickness of each paint system, both before and during exposure, are shown in table 4. These data were obtained only for the test panels in exposures A, B, and C. Similar data were not recorded for exposures D and E as existing paint, where intact, had not been removed from the bridge member before application of the test paint. Consequently, accurate measurements of the applied test paints were not

Table 3.—Cumulative numerical analysis of panel ratings for general appearance, erosion, and rusting at 23 months' exposure

Paint System	Total ratings of panels for exposures A through E			
	General appearance	Erosion	Rusting	Cumulative total
1	33	33	40	106
2	38	38	41	117
3	29	32	36	97
4	9	11	25	45
5	36	41	39	116
6	6	[1] 2+	17	[1] 25+
7	36	34	35	105
8	44	47	50	141
9	25	24	32	81

[1] Erosion rating for Paint System 6 in exposure E was not made and therefore is not included in totals.

Table 4.—Decrease in film thickness in relation to length of exposure

Paint System	Average film thickness, mils						
	Before exposure	After exposure by months					
		11	15	23	33	35	38
EXPOSURE A (MILE 27 BRIDGE, NORTH EXPOSURE, 1 PANEL EACH)							
1	6.5	0.0	0.0	0.0	0.0	0.0	0.0
2	3.5	0.2	0.0	0.0	0.0	0.0	0.0
3	[1] 1.5(4.5)	0.0	0.0	0.0	0.0	0.0	0.0
4	3.8	1.5	0.0	0.0	0.0	0.0	0.0
5	[1] 3.0(5.0)	4.0	2.1	1.9	[2] 0.4	[2] 0.2	[2] 0.2
6	3.5	0.0	0.0	0.0	0.0	0.0	0.0
7	4.0	2.2	1.7	[2] 0.4	0.0	0.0	0.0
8	18.0	13.5	9.5	9.1	8.8	8.7	8.7
9	2.0	0.4	0.0	0.0	0.0	0.0	0.0
EXPOSURE B (MILE 28 BRIDGE, NORTH EXPOSURE, 4 PANELS EACH)							
1	6.5	6.5	5.3	4.9	4.0	3.6	3.5
2	3.5	3.4	2.4	1.8	2.1	1.8	1.6
3	[1] 1.5(4.5)	3.0	1.8	1.6	1.6	1.2	1.0
4	3.8	1.1	0.7	0.0	0.0	0.0	0.0
5	[1] 3.0(5.0)	4.6	3.7	3.0	3.0	4.1	2.4
6	3.5	2.6	2.3	2.8	2.1	----------	2.0
7	4.0	1.8	1.8	2.4	1.7	----------	1.5
8	18.0	15.0	14.6	11.2	8.8	----------	9.0
9	2.0	2.2	1.4	2.1	1.1	----------	1.0
EXPOSURE C (MILE 28 BRIDGE, PROTECTED SOUTH EXPOSURE, 1 PANEL EACH)							
1	6.5	4.5	4.0	4.2	3.9	----------	4.0
2	3.5	2.5	1.6	1.3	1.9	----------	1.8
3	[1] 1.5(4.5)	4.8	3.9	4.6	3.9	----------	3.6
4	3.8	3.0	1.8	1.9	1.1	----------	1.7
5	[1] 3.0(5.0)	4.5	2.5	2.9	2.6	----------	2.5
6	3.5	3.0	2.0	1.6	1.8	----------	1.7
7	4.0	2.0	1.1	1.0	1.0	----------	1.0
8	18.0	18.0	13.6	15.0	10.6	----------	12.0
9	2.0	2.2	1.0	0.9	0.9	----------	0.9

[1] These measurements of film thickness are very questionable as subsequent readings, especially in the milder exposures B and C, were substantially more, which meant that the paint was thicker than recorded originally. Consequently, the values shown in parentheses were obtained by extrapolation to zero exposure time and probably are measurements closer to the true original thicknesses.

[2] Measured on irregular areas where paint remained between eroded and rusted patches.

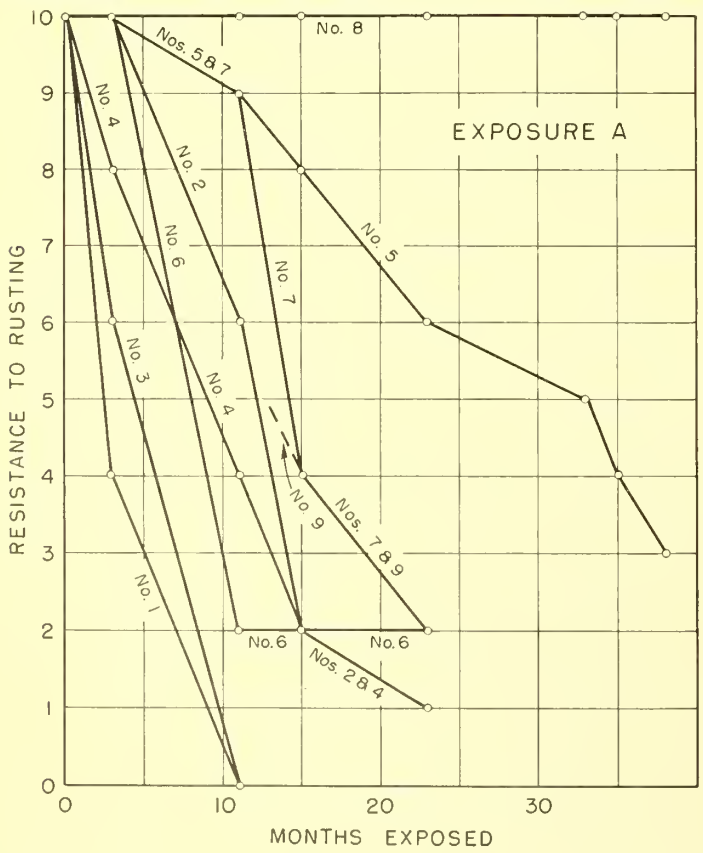

Figure 17.—Decrease in resistance to rusting at exposure A in relation to duration of exposure.

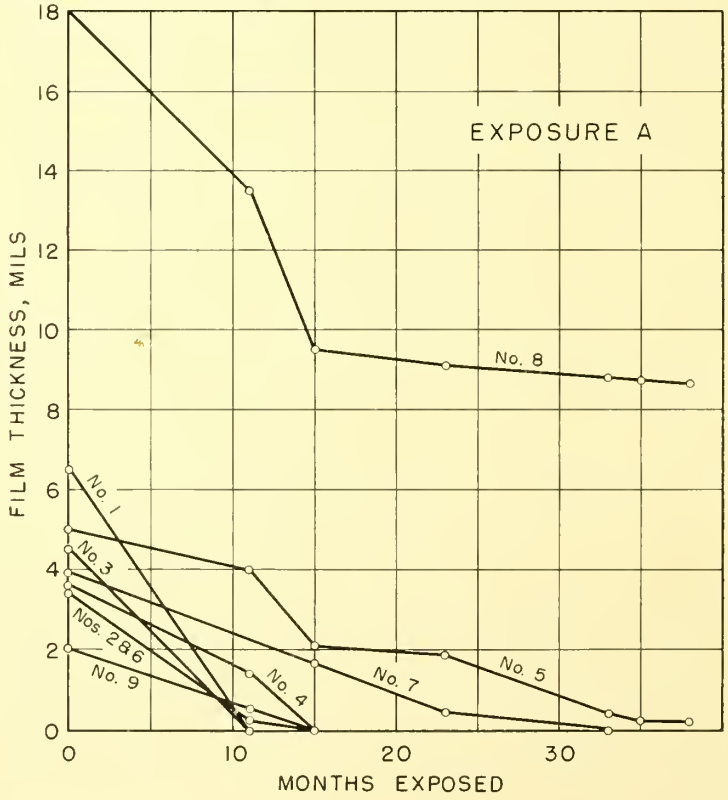

Figure 18.—Decrease in film thickness of each Paint System during exposure A.

Table 5.—Analysis of Paint System 1

Paint System 1	Washcoat coating, 1-1			Primer 1-2	Finish 1-3
	Resin component	Acid catalyst component	Mixture [1]		
General properties:					
Color			Olive, clear	Black	Black
Weight per gallon—pounds	7.1			11.0	10.3
Viscosity—Krebs units	72			ca. 141	95
Drying time:					
Set to touch—hours			1/60	1	½
Dry through—do			1/10	<24	1
At dry film thickness—mils				4.4	4
Composition, general:					
Pigment—pct. by wt	6.2	None		None	
Volatile—do	84.7	84.1		55.8	47.1
Nonvolatile vehicle—do	9.1	15.9		44.2	52.9
Nature of pigment	Zinc chromate with perhaps some magnesium silicate.	None			
Volatile:					
Nature, from infrared analysis	Isopropanol	Ethyl alcohol and water.		Aromatic	Water
Nonvolatile vehicle composition:					
Nature, from infrared analysis	Polyvinyl butyral resin.	Phosphoric acid		Coal tar	Coal tar
General nature	[2]	[2]	[2]	Coal tar cutback.	Coal tar emulsion.

[1] Mixture of 1 part catalyst to 6 parts resin component, by volume.
[2] Washcoat 1-1 was basic zinc chromate vinyl butyral.

possible. Consideration of only the data on exposure A (table 4)—because the potential performance of the test paints were best sorted out in this harsh exposure—shows that the film thickness remaining after 23 or even 38 months of exposure reinforces the previous findings that paint Systems 5 and 8 were far superior to the others and that No. 8 furnished a very substantial amount of protection even after 38 months of exposure. These same data for exposure A are shown plotted in figure 18.

From data visual in figure 18, it is again apparent that paint System 8 will retain a satisfactory amount of film protection far beyond 38 months. The flat slope of the line (fig. 18) for System 8 from 15 to 38 months suggests a somewhat long future life for this paint. An expectation that the relative overall slopes of each line in figure 18 might have been used to predict the ultimate performance of each paint had they all been applied at equal thicknesses was not fulfilled. However, data plotted for Systems 1, 2, 3, and 6 provided steeper slopes overall, indicating generally that these paints were inferior to the rest of the paint systems. Data for the other paints are represented by flatter slopes, thus indicating that these paints might provide longer periods of protection if applied in thicker films.

Table 6.—Analysis of Paint System 2

Paint System 2	Primer coating, 2-1	Finish coating, 2-2
General properties:		
Color	Gray	Black
Weight per gallon—lbs	13.9	8.5
Viscosity—Krebs units	54	61
Drying time:		
Set to touch—hours	6	Within 24
Dry through—do	Within 24	Within 36
At dry film thickness—mils	2.6	3.9
Composition, general:		
Pigment—pct. by wt	59.4	14.5
Volatile—do	12.3	8.5
Nonvolatile vehicle—Pct. by wt	28.3	77.0
Pigment composition:		
Basic carbonate white lead—percent	39.8	
Siliceous matter—do	[1] 26.9	
Zinc oxide—do	18.8	
Calcium carbonate—do	14.4	
Carbon black—do		81.2
Lead (Pb_3O_4)—do		8.4
Iron oxide (Fe_2O_3)—do		0
Nonvolatile vehicle composition:		
Iodine number of extracted fatty acids	117	181
Nature, from infrared analysis	[2]	[3]
General nature	[4]	[5]

[1] Includes quartz.
[2] Essentially a modified drying oil (such as linseed) with other oils and/or resin.
[3] Essentially a processed or limed drying oil (such as linseed).
[4] White lead and extender pigment, in modified drying oil.
[5] Carbon black and red lead in modified drying oil.

Laboratory Evaluation

As mentioned previously, a laboratory evaluation of the paint systems, as a supplement to the field studies, initially had not been considered. The laboratory work was undertaken after the field study was well underway. The purposes of this supplementary investigation were twofold. First, as complete an analysis of each paint as possible was desirable to generically define each paint and perhaps relate such information to performance and, at the same time, obtain a basis for preparing suitable specifications for the ultimate use of any promising paint system. Second, in the field tests the paints were applied in the film thicknesses recommended by the paint supplier or as normally applied in conventional practices. The field study, therefore, was limited to a comparison of paints of different film thicknesses. As film thickness may have contributed greatly to abrasion resistance, especially System 8, a decision was made to explore in accelerated laboratory tests the inherent abrasion resistance of each paint by the use of controlled film thicknesses. Thus the effect of film thickness could be eliminated or accounted for. This phase of the laboratory work was considered very important, as one of the systems may have had a strong advantage over others in the field study because it was applied more thickly. The laboratory work was conducted in the laboratories of the Bureau of Public Roads, Washington, D.C., and the findings are described in the following paragraphs.

Nature and Composition of the Paints

Complete physical, chemical, and infrared spectral analyses of each paint system were made and the results are detailed in tables 5 through 13. The methods used for chemical analysis and physical examination generally were those described by the Federal Test Method for paints (*2*). Infrared spectral analysis was conducted, where necessary, to generically identify each vehicle and volatile thinner. The spectroscopic methods used have been described previously in an issue of the Official Digest (*3*). Tables 5 through 13 contain all the detailed data on composition, but some brief and amplifying remarks on these findings are given in the following paragraphs for each of the paint systems.

System 1, coal tar

System 1 consisted of a washcoat, a primer, and a finish coat. The washcoat was a zinc chromate vinyl butyral coating; the primer was an unpigmented coal tar cutback; and the finish coat was a coal tar emulsion. Other than the washcoat, no rust-inhibitive pigment was present in this system.

Table 7.—Analysis of Paint System 3

Paint System 3	Primer coating, 3-1	Finish coating, 3-2
General properties:		
Color	Brown	Orange
Weight per gallon—pounds	11.9	17.0
Viscosity—Krebs units	53	66
Drying time:		
Set to touch—hours	2	¾
Dry through—do	6	5
At dry film thickness—mils	2.0	1.9
Composition, general:		
Pigment—pct. by wt	52.7	65.1
Volatile—do	30.7	17.5
Nonvolatile vehicle—do	16.6	17.4
Pigment composition:		
True red lead (Pb_3O_4)—pct	48.9	84.8
Siliceous matter—do	15.7	[1] 9.2
Zinc oxide—do	11.6	
Iron oxide (Fe_2O_3)—do	9.0	
Zinc chromate ($ZnCrO_4$)—do	7.3	
Nonvolatile vehicle composition:		
Phthalic anhydride—percent	30.1	19.5
Oil acids—do	55.5	65.5
Nature, from infrared analysis	[2]	[3]
General nature	[4]	[5]

[1] Essentially talc.
[2] Medium oil-alkyd resin.
[3] Long oil-alkyd resin.
[4] Mixed pigment (red lead, siliceous matter, zinc oxide, zinc chromate, iron oxide) in alkyd vehicle.
[5] Red lead in alkyd vehicle.

Table 8.—Analysis of Paint System 4 [1]

Paint System 4	Finish coating, 4–2
General properties:	
Color	White
Weight per gallon ... pounds	8.2
Viscosity ... Krebs units	94
Drying time:	
Set to touch ... hours	3
Dry through ... do	8
At dry film thickness ... mils	1.2
Composition, general:	
Pigment ... pct. by wt	11.1
Volatile ... do	74.1
Nonvolatile vehicle ... do	14.8
Pigment composition:	
Titanium dioxide ... percent	95.3
Nonvolatile vehicle, composition:	
Nature from infrared analysis	Vinyl chloride-vinyl acetate copolymer with perhaps some polyvinyl alcohol.
General nature	Titanium dioxide vinyl copolymer paint.

[1] Washcoat 4–1 for System 4 was the same as used for System 1, as shown in table 5.

System 2, drying oil

System 2 consisted of a primer and finish coat, both of a drying-oil type. The primer pigment was not rust inhibitive, containing only a mixture of white lead and extender pigments, and the finish coat was pigmented primarily with carbon black but also contained a small amount of red lead. Therefore, System 2 was not typically rust inhibitive.

System 3, alkyd

System 3 consisted of a primer and finish coat of the oil alkyd type. The primer pigment contained both red lead and zinc chromate, which were substantially extended by other pigments, and the finish coat was pigmented essentially with red lead. A red lead alkyd paint such as this normally is not used as a finish coat because of its poor weathering properties. Evidently, it was incorporated in this system only upon the recommendations of the supplier and without prior consideration having been given to details of its pigmentation. This entire paint system was intended to be, and was represented by the supplier to be, a phenolic system; actually it was an alkyd system as determined by these analyses, which included infrared spectroscopy.

Table 9.—Analysis of Paint System 5

Paint System 5	Primer coating, 5–1	Finish coating, 5–2
General properties:		
Color	Brown	White
Weight per gallon ... pounds	13.9	8.5
Viscosity ... Krebs units	76	47
Drying time:		
Set to touch ... hours	¼	1
Dry through ... do	1	6
At dry film thickness ... mils	3.2	1.8
Composition, general:		
Pigment ... pct. by wt	58.7	19.8
Volatile ... do	22.4	57.4
Nonvolatile vehicle ... do	18.9	22.8
Pigment composition:		
True red lead (Pb_3O_4) ... pct	48.8	------
Siliceous material ... do	16.9	------
Iron oxide (Fe_2O_3) ... do	8.0	------
Zinc oxide (ZnO) ... do	1.4	------
Titanium dioxide (TiO_2) do	------	91.1
Zinc chromate ($ZnCrO_4$) do	10.1	------
Nonvolatile vehicle:		
Nature, from infrared analysis	(1)	(1)
General nature	(2)	(3)

[1] Epoxy ester.
[2] Mixed pigment (red lead, siliceous matter, iron oxide, zinc oxide, zinc chromate) in epoxy ester vehicle.
[3] Titanium dioxide epoxy ester paint.

System 4, vinyl

System 4 consisted of a vinyl butyral washcoat, as used in System 1, and a finish coat of titanium dioxide pigmented vinyl paint, most likely a copolymer of vinyl chloride and vinyl acetate. Except for the washcoat, the system did not contain rust-inhibitive pigmentation.

System 5, epoxy ester

System 5, although originally represented as an epoxy, consisted of an epoxy ester primer and finish coat. The primer contained both red lead and zinc chromate extended with other pigments, and the finish coat was pigmented with titanium dioxide.

System 6, zinc-rich inorganic

System 6, typical of many commercially available post-cured zinc-rich silicate paints, consisted of a two-package zinc and sodium silicate coating that is post cured with a solution of phosphoric acid after it has been mixed and applied.

System 7, neoprene

System 7 consisted of a primer and finish coat, both neoprene based. The primer did not contain pigmentation, and the finish coat was pigmented essentially with carbon black. The system did not contain rust-inhibitive pigments.

System 8, polysulfide

System 8 consisted of a primer of an unpigmented chlorinated rubber solution and a finish coat that was a two-package catalyzed liquid polysulfide rubber. This system was referred to as a rubber coating by the supplier and did not contain a rust-inhibitive primer.

System 9, aluminum varnish

System 9 consisted of a primer, which was an extended red lead alkyd paint, the same as in System 3, and a finish coat of a ready-mixed aluminum oleoresinous varnish. This system was intended to represent the standard paint system in general use in Alaska, but it was not exactly the same because of the over-extended red lead pigment, alkyd rather than drying oil in the primer, and the fact that the finish coat was a ready-mixed aluminum paint rather than a two-package system mixed just prior to its use.

Laboratory abrasion study

The method used for the laboratory abrasion test is that given by ASTM Method D 658–44 (*1*). It was selected because the mode of abrasive action seemed to approximate the special condition in Alaska more closely than other available abrasion tests. Briefly stated, the test is conducted on a painted panel of known film thickness that is exposed to silicon carbide abrasive particles impelled by a constant air pressure and airflow. After the coating has been worn through to the metal substrate, the weight of the abrasive particles consumed in the test is determined and divided by the original thickness of the coating. The calculated result is called an abrasion coefficient whose units are grams per mil. The larger this coefficient, the more resistant the coating to abrasive forces. More specific details of the procedure used are given in the following paragraphs and in reference *1*.

Only the finish coat of each paint system was included in this test, as it is this coat that must resist abrasion and protect the underlying primer, when present, from exposure and wear. Controlled thicknesses of each such finish coating were applied by an applicator blade to clean, tin-coated panels. After being aged for a few months to achieve adequate and maximum film curing, the coated panels were then evaluated for abrasion resistance. For comparison, auxiliary tests on fresh films were conducted on several of the more promising coatings to more closely approximate the practical field conditions, where a newly painted surface might be exposed to abrasive forces.

The coated panels were measured for film thickness by both a micrometer and a magnetic gage. Each test panel was then mounted at a 45° angle in a Gardner Grit Blast Abrasimeter, as shown in figure 19. A schematic of the entire assembly is shown in the referenced ASTM Method (*1*). The hopper, as

Table 10.—Analysis of Paint System 6

Paint System 6	Finish coating, 6–1 [1]		Acid cure solution 6–2
	Powder	Solution	
General properties:			
Color	(2)	(2)	Colorless
pH	------	--------	1.9
Composition:			
Metallic zinc (Zn) ... pct. by wt	83.5	--------	---------
Zinc oxide (ZnO) ... do	3.5	--------	---------
Lead (Pb) ... do	12.3	--------	---------
Siliceous material (as SiO_2) ... do	------	21.8	---------
Sodium oxide (Na_2O) ... do	------	7.0	---------
Phosphoric acid (H_3PO_4) ... do	------	--------	23.0
Volatile solvent pct. by wt	None	62.9	50.5
Nature of solvent from infrared analysis	------	Water	(3)
General nature	(4)	(4)	(5)

[1] Components mixed for use in proportion of 23 pounds powder to ⅔ gallon of solution.
[2] Color of cured film of mixture of powder and solution was gray.
[3] Mixture of alcohol and ketone (probably isopropanol and acetone).
[4] Post-cured inorganic zinc-rich silicate coating.
[5] Phosphoric acid-alcohol-ketone curing solution.

Table 11.—Analysis of Paint System 7

Paint System 7	Primer coating, 7-1 [1]	Finish coating, 7-2
General properties:		
Color	Amber, clear	Black
Weight per gallon—lbs.	(2)	8.1
Viscosity—Krebs units	(2)	121
Drying time:		
Set to touch—hours	(2)	⅛
Dry through—do	(2)	2
At dry film thickness mils	(2)	2.0
Composition, general:		
Pigment—pct. by wt.	None	17.5
Volatile—do	79.2	63.5
Nonvolatile vehicle—do	20.8	19.0
Pigment composition:		
Carbon black—percent	----------	[3] 88
Siliceous material—do	----------	[3] 12
Nonvolatile vehicle composition:		
Nature, from infrared analysis	(4)	(5)
General nature	(6)	(7)

[1] Material received was viscous and may have lost some solvent before analysis.
[2] Too viscous for test.
[3] Approximate value.
[4] Mixture of neoprene and possibly a phenolic resin.
[5] Neoprene.
[6] Neoprene solution.
[7] Carbon black neoprene.

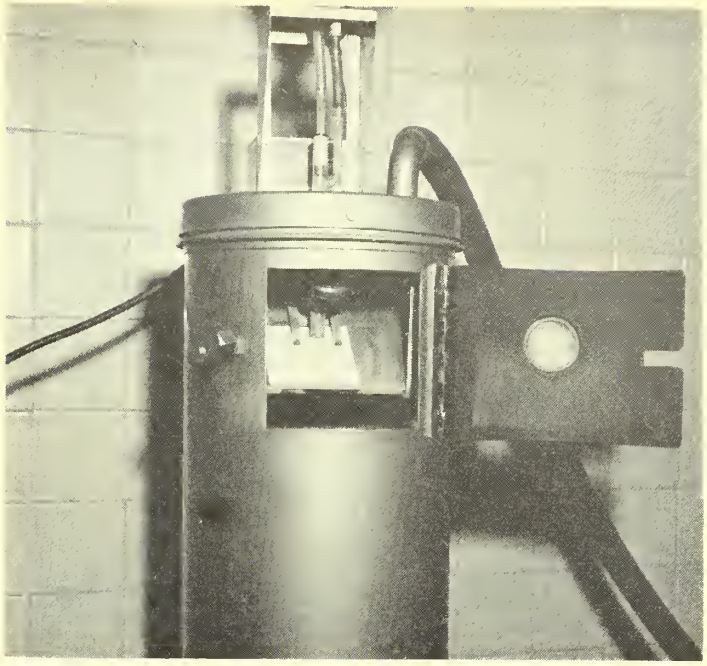

Figure 19.—Test panel mounted in the Grit Blast Abrasiometer.

Table 12.—Analysis of Paint System 8

Paint System 8	Primer coating, 8-1	Finish coating, 8-2 [1]	
		Base	Accelerator
General properties:			
Color	Colorless	Black	Brown
Weight per gallon pounds	7.9	11.4	25.3
Viscosity Krebs units	(2)	(2)	(2)
Drying time:			
Set to touch hours	⅛	------	------
Dry through hours	½	------	------
At dry film thickness mils	1.3	------	------
Composition, general:			
Pigment pct. by wt.	None	31.5	72.5
Volatile—do	73.3	11.1	5.1
Nonvolatile vehicle—do	26.7	57.4	22.4
Pigment composition:			
Carbon black percent	--------	75.3	------
Calcium carbonate—do	--------	22.2	------
Lead dioxide (PbO_2)—do	--------	------	76.1
Nonvolatile vehicle composition:			
Nature, from infrared analysis	(3)	(4)	(5)
General nature	(6)	(7)	(7)

[1] Mixed in proportion of 10 parts base+1 part accelerator by weight.
[2] Too viscous for test.
[3] Chlorinated rubber.
[4] Liquid polysulfide resin.
[5] Styrene polymer or copolymer.
[6] Chlorinated rubber solution.
[7] General nature for finish coating 8-2 was lead oxide catalyzed liquid polysulfide rubber.

Table 13.—Analysis of Paint System 9 [1]

Paint System 9	Finish coating, 9-2
General properties:	
Color	Aluminum
Weight per gallon—pounds	7.9
Viscosity—Krebs units	44
Drying time:	
Set to touch—hours	2½
Dry through—do	Within 40
At dry film thickness—mils	1.9
Composition, general:	
Pigment—pct. by wt.	11.1
Volatile—do	46.1
Nonvolatile vehicle—do	42.8
Pigment composition:	
Nature	Essentially aluminum.
Nonvolatile vehicle composition:	
Iodine number of extracted fatty acids	107
Nature, from infrared analysis	Modified drying oil varnish.
General nature	Ready-mixed aluminum oleoresinous varnish paint.

[1] Primer 9-1 for System 9 was the same as used for System 3, as shown in table 7.

shown in figure 20, was loaded with a weighed amount of silicon carbide, grain size 180, type GG, which previously had been sieved as required by the test method, and the abrasive particles were admitted into the test chamber. These abrasive particles were impelled by the airflow and impinged upon the coating at a 45° angle. The flow of abrasive particles was stopped at a point when the coating had been worn through to the substrate to the degree required by the method. This condition was monitored through a glass window in the test apparatus (fig. 20). A typical condition of a test coating showing the abraded spot at the conclusion of the test is shown in figure 21. The weight of abrasive particles consumed in the test was determined, and the abrasion coefficient was calculated.

The results of the laboratory abrasion tests are shown in table 14 in terms of abrasion coefficient, where each value shown represents the average of two or more determinations. Generally, agreement between individual test results was good. Because the abrasion coefficient is usually considered to be independent of film thickness, a test of a single coating thickness is normally sufficient. However, to assess this belief more fully, tests were conducted on more than one coating thickness. An attempt was also made to include or bracket the thickness of finish coating used in the field study. For System 8, however, it was not practical to conduct additional tests at the 15- to 20-mil film thicknesses used in the field study. A 1½-mil coating of this material had such an unusually high abrasive resistance, and a single test consumed so much time and abrasive material, that it was considered impracticable to conduct additional tests at the 15- to 20-mil thicknesses.

Several things are evident from the abrasion results shown in table 14. First, the abrasion coefficient was not always completely independent of the initial film thickness of the coating. Allowing for the known reproducibility and variations of the test method, it is apparent that in several systems the abrasion coefficient was higher for thicker films of the same material. Several explanations of this

Figure 20.—Abrasiometer, door closed during test, and hopper at top.

Table 14.—Laboratory abrasion test results [1]

System coating tested	Dry film thickness [2]	Abrasion coefficient [3]
	Mils	
1–3	2	5
	4	5
	10	6
2–2	1	47
	1½	76
	3	116
3–2	1½	18
	2	24
4–2	1	49
	2	56
5–2	1	40
	2	54
6–1, 2	2½	[4] 130
7–2	½	[5] 750
	1½	860
	2	990
8–2	1½	1, 155
		[6] 2, 100
9–2	1½	18
	3½	29

ASTM Method D 658-44, *Standard Method of Test for Abrasion Resistance of Coatings of Paint, Varnish, Lacquer and Related Products with the Air Blast Abrasion Tester.* Finish coatings only were applied to tin panels and unless otherwise stated were aged 4 to 5 months before test. Each result is an average of two or more determinations.

[2] Precise film thickness measurements were made and used in the calculation of the abrasion coefficient. For convenience, results were grouped under the nominal film thickness shown in the table that most closely approximated the actual thickness.

[3] Weight in grams of carborundum used per mil of film thickness to wear through film.

[4] The reliability of the results for this particular coating is highly doubtful because of the unusual difficulty in visually observing a clear-cut endpoint.

[5] Tested on fresh film, 20 hours old.

[6] Tested on fresh film, 24 hours old.

are possible; such as, greater retention of impinging abrasive particles by thicker films provide a buildup of a more abrasive-resistant armor coating; or else, greater elasticity of thicker films because of the greater retention of solvents, plasticizers, or perhaps less overall embrittlement throughout the thicker film because of more limited oxidation. These aspects were not investigated further.

Another and more important observation that can be made from table 14 data is that the abrasive resistance qualities of Systems 7 and 8 exceeded by far those of the other paints when films of comparable thickness were compared. Of these two, No. 8 exhibited the most abrasion resistance, having an abrasion coefficient of 1,155. This system exceeded the abrasion resistance of the poorest system (No. 1) by a tremendous factor (more than 200 times) and exceeded that of the so-called standard system (No. 9) by a factor of 65. This latter result might be extrapolated to mean that if System 9 lasted only 6 months in a severe abrasive atmosphere, then System 8 should theoretically last 65 times as long, or some 30 years. This extrapolation would obtain provided no other degradation conditions were operative, such as film oxidation or loss of adhesion. Obviously, other degradation factors are always present to affect the life of a paint. It is also apparent from the results shown in table 14 that, should a fresh film of the finish coating of System 8 be exposed to an abrasive environment, the abrasion resistance of such a paint would not be affected to any large extent by an early exposure and its resistance might even be increased.

In summary, the relative order of abrasion resistance in the laboratory tests, with the coatings ranked from best to poorest, is as follows:

$$8>7>6>2>4,\ 5>3,\ 9>1.$$

Both of the better Systems, 7 and 8, were rubberlike systems; that is, neoprene and polysulfide, respectively. Interestingly, System 9, which had relatively poor resistance, is similar to the standard paint normally used in Alaska, and the one that caused all the difficulty in the painting of the reconstructed highway bridges.

Field and Laboratory Results Compared

Both the field and laboratory results are in good agreement that the polysulfide paint, System 8, had by far the most abrasion resistance of any of the paint systems tested. The excellent abrasive resistance of a thin film of this material in the laboratory test clearly showed that the field study conclusions were not seriously biased by the use of thicker films of this coating as compared to the other coatings studied. The excellent abrasive resistant properties of the polysulfide coating is in accord with unpublished findings of A. G. Roberts of the National Bureau of Standards. These latter findings were based on both service performance and a laboratory NBS Abrasive Jet Method that is similar but much more intense than the ASTM test used in this work. Details of the NBS abrasive test have been described by Roberts (*4*).

The lack of agreement between the laboratory and field study findings relative to System 7, the neoprene paint, were somewhat disappointing. The laboratory results indicated that this System has excellent abrasion resistance. This was also true according to findings of Mr. Roberts of the Bureau of Standards. Yet the field exposure study did not show, in any dramatic sense, the superiority of this coating as it did for the polysulfide coating. A possible explanation for this discrepancy is that the neoprene system (No. 7) used in the field did not contain a rust-inhibitive primer, and that the relatively thin coating used for this system may have permitted the passage of moisture and consequent undercutting and failure of the paint by corrosion of the underlying metal. As the backs of the test panels were not coated, undercutting of this film by corrosion products was in fact a real possibility.

It will be recalled that the epoxy ester system (No. 5) ranked next to polysulfide (No. 8) in affording protection in field exposure A, the more severe exposure. Yet the laboratory abrasion tests did not reveal any substantial abrasive-resistant properties for this

Figure 21.—Test panel after laboratory abrasion test, showing two test areas where coating was worn through to bare metal.

system. This discrepancy can perhaps be explained as before, on the basis that System 5 was in a favored position in field exposure A. As indicated previously in this article, System 5 was so located in field exposure A that it received the least wind action of the nine coatings tested.

The evidence from both the field and laboratory results showed that the remainder of the paints, including the presumably standard system (No. 9), did not offer any promise for satisfactory performance for the type of exposure conditions existing in the Copper River Delta.

ACKNOWLEDGMENT

The early work of the Bureau of Public Roads former Materials Section in the Anchorage Division office is acknowledged, especially because this office recognized and defined the problem and then planned and conducted much of the field investigation before Alaska acquired statehood. The author is also indebted to the Planning and Research Section of the Alaska Department of Highways for its efforts in completing the field study started by Public Roads and for making available pertinent data, information, and reports relating to the field exposure study. The laboratory work of Ray Cherwinski in conducting much of the chemical and infrared analysis of the paint systems is also acknowledged.

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(*3*) *Infrared Spectroscopy: Its Use as an Analytical Tool in the Field of Paints and Coatings,* by the Infrared Spectroscopy Committee of the Chicago Society for Paint Technology, Official Digest of the Federation of Societies for Paint Technology, vol. 33, No. 434, Part 2, March 1961.

(*4*) *Improved NBS Abrasive Jet Method for Measuring Abrasion Resistance of Coatings,* by A. G. Roberts, ASTM Bulletin No. 244, February 1960, pp. 48–51.

Voids, Permeability, and Film Thickness Related to Asphalt Hardening

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

Reported by [1] JOSEPH F. GOODE, Highway Research Engineer, and LAWRENCE A. LUFSEY, Highway Engineering Technician, Materials Division

The possibility of using air permeability measurements as control factors in the design of bituminous mixtures was investigated during the study reported in this article. Although air permeability as a possible control factor in bituminous pavement compaction also is of interest to highway engineers, this investigation was limited only to the effect of air permeability on the composition of bituminous mixtures.

The authors obtained no confirmation that air permeability measurements should be a factor in the design of bituminous mixtures. They did, however, determine that a criterion based on a combination of (1) the thickness of the asphalt coating on the aggregate particles and (2) the air void content of the bituminous mixture should provide a design for a more serviceable mixture. Bituminous mixtures designed according to this criterion should not be so subject to early pavement failure caused by excessive hardening of the asphalt during service.

Introduction

MOST bituminous paving engineers and technologists agree that high air permeability, high content of air voids, and thin bituminous coatings on the aggregate particles should be avoided in the design and construction of bituminous pavements. They also agree that a low permeability is required for the pavement to serve as a protective cover for the underlying courses. Many believe that the rate at which air can flow through a pavement affects the rate of hardening of the bituminous material and that high permeability is therefore undesirable. It has long been recognized that a high proportion of air voids and thin bituminous coatings can be contributing causes of an excessively high rate of hardening of the bituminous material and the subsequent early failure of the pavement surface.

Nicholson (*1*),[2] for example, stated in a 1937 AAPT paper, ". . . it follows that the less voids there are in a given paving mixture, the less the action of air on the asphalt." In another paper presented at the same meeting, Hubbard and Gollomb (*2*) concluded: ". . . Rapidity of hardening is a function of film thickness, temperature and time, . . ." These authors also concluded: ". . . To insure long life, prevent cracking and possible disintegration in an asphalt pavement or wearing course, due to hardness of the asphalt, the following . . . rules should be strictly observed: . . . Use as high a percentage of asphalt as possible without reducing stability below the minimum required to prevent displacement under traffic. In this way film thickness is increased to the maximum practical extent and air permeability decreased. . . . Compress all asphalt mixtures thoroughly so that they will be (as) impermeable to air as possible."

More recently, in a 1958 AAPT paper, Heithaus and Johnson (*3*) showed that the degree of asphalt hardening increases strongly as the initial air void content of the pavement increases. Although permeability was not measured, the authors attributed the increase in rate of hardening to the increase in air permeability, which was assumed to be a function of voids. In 1961, Goode and Owings (*4*) reported the results of a laboratory-field study showing that high content of initial air voids tends to cause rapid hardening of the asphalt and early deterioration of the pavement surface. Air permeability and asphalt film thickness were not evaluated in that study and could have been factors affecting asphalt hardening and pavement performance. The number of other research studies on the effect of air void content on pavement characteristics and performance is too large for inclusion as references in this article.

References to research on the effect of asphalt film thickness on rate of hardening are very limited. Probably the most important research was reported at the 1959 meeting of the Association of Asphalt Paving Technologists by Campen, Smith, Erickson, and Mertz (*5*). These authors established minimum and maximum values of film thickness for satisfactory pavement performance.

Research employing air permeability measurements also has been somewhat limited. Ekse and Zia (*6*), at the 1953 meeting of AAPT, described a method of measuring air permeability to be used in control of pavement compaction. In 1955 McLaughlin and Goetz (*7*) reported on the use of a special device for measuring air permeability. Results of this study showed good relationships between permeability and air voids for a particular mixture but none between permeability and changes in mixture properties caused by freezing and thawing. Asphalt hardening was not a variable in the study.

In a 1960 report, Ellis and Schmidt (*8*) described a new simple device for measuring air permeability of laboratory samples and of pavement in situ. Since the publication of this report and a subsequent report by Hein and Schmidt (*9*) considerable attention has been directed toward air permeability measurements. Kari and Santucci (*10*) presented a paper on the subject at the February 1963 meeting of AAPT and Warner and Moavenzadeh (*11*) at the June 1964 meeting of the American Society for Testing and Materials (ASTM).

None of these referenced studies related air permeability to asphalt hardening. A pavement having high air permeability might be expected to show early evidence of distress caused by rapid hardening of the asphalt. But, high permeability could be the result of one or more of these factors: (1) insufficient asphalt in the mixture or inadequate compaction accompanied by a high percentage of air voids in the mixture; or (2) the use of an aggregate gradation that has an excessively high surface area accompanied by a thin film of asphalt coating on the aggregate particles. The actual cause of the rapid hardening of the asphalt could be high air void content, thin asphalt film, or a combination of the two. No data have been provided to prove that air permeability per se is a factor that affects the rate of asphalt hardening where aggregate gradation is a variable. All three factors might contribute to the rate of asphalt hardening.

[1] Presented at the annual meeting of the Association of Asphalt Paving Technologists, Philadelphia, Pa., February 15–17, 1965.

[2] References indicated by italic numbers in parentheses are listed on page 222.

Table 1.—Properties of asphalt

Original asphalt:		
Penetration	77° F., 100 g., 5 seconds	91
Saybolt furol viscosity	275° F., second	190
Specific gravity	77°/77° F.	1.018
Flash point	° F.	625
Soluble in CCl_4	percent	99.86
After standard oven test:		
Change in weight	percent	+0.01
Penetration of residue	77° F., 100 g., 5 seconds	87
Retained penetration	percent	96
After thin film oven test:		
Change in weight	percent	+0.08
Penetration of residue	77° F., 100 g., 5 seconds	62
Retained penetration	percent	68
Ductility of residue	77° F., 5 cm./min., cm	250+

Table 2.—Properties of aggregate components and aggregate blends

Properties	Crushed granite	Natural sand			Limestone dust
		No. 1	No. 2	No. 3	
AGGREGATE COMPONENTS					
Apparent specific gravity	2.82	2.67	2.67	2.66	2.71
Bulk specific gravity	2.75	2.58	2.58	2.63	[1] 2.71
Aggregate passing sieve:					
½-inch, percent	100				
No. 8, do	0	100			
No. 16, do	0	63		100	
No. 30, do	0	9	100	99	
No. 50, do	0	1	51	88	100
No. 100, do	0	1	25	19	97
No. 200, do	0	0.4	9.1	1.0	89.1
Aggregate by weight in:					
Blend A, percent	41	31	6	9	13
Blend B, do	47	28	5	8	12
Blend C, do	53	25	5	7	10
Blend D, do	59	22	4	6	9
Blend E, do	65	19	3	5	8

	A	B	C	D	E
AGGREGATE BLENDS					
Aggregate passing sieve:					
½-inch, percent	100	100	100	100	100
⅜-inch, do	91	89	88	87	85
No. 4, do	73	68	64	60	56
No. 8, do	59	53	47	41	35
No. 16, do	48	43	38	33	28
No. 30, do	31	27	24	21	18
No. 50, do	24	22	19	17	14
No. 100, do	16	15	13	11	10
No. 200, do	12.3	11.3	9.5	8.5	7.5
Apparent specific gravity [2]	2.734	2.743	2.751	2.760	2.769
Effective specific gravity	2.715	2.726	2.738	2.749	2.761
Bulk specific gravity [2]	2.669	2.677	2.685	2.693	2.702
Surface area ft.² [2] per lb.	50.5	46.4	40.3	35.7	31.5

[1] Not measured but assumed to be same as apparent specific gravity.
[2] Computed from preceding data in table.

Objectives of Study

The objectives for the study reported in this article were to determine: (1) the best procedure for measuring air permeability of laboratory compacted specimens; (2) whether air permeability was of sufficient importance to be a part of a mix design procedure; (3) the relative effect on the rate of asphalt hardening of air voids, air permeability, and thickness of the asphalt film coating and particles; and (4) whether the exponent 0.45 used by the Bureau of Public Roads in setting up its Gradation Chart (*12*) was the most suitable value for use with crushed stone aggregate.

Conclusions

Based on the limited number of materials, mixtures, and aging conditions used in the study reported in this article on resistance to asphalt hardening, the following conclusions appear justified:

- The exponent 0.45 used in establishing the BPR Gradation Chart—proved to be satisfactory for use with gravel mixtures in another study (*12*)—is equally satisfactory for use with mixtures containing crushed stone as the coarse aggregate. As in the other study, test results indicated that 0.435 might be a better exponent but the actual differences in results are not considered practically significant.
- When measuring air permeability, the vertical face of the asphalt specimen should be sealed to prevent leakage of air between the side of the specimen and the rubber membrane of the testing mold. When a seal is not used, test results will be excessively high, especially when air void contents are less than 7 percent and mixtures are coarse-grained. Paraffin is a satisfactory sealing material but should not be used on specimens that are to be extracted for tests on recovered asphalt.
- Air permeability is a function of aggregate gradation as well as of air void content. The effect of differences in gradation is much more pronounced when air void content is high.
- When the mix is designed to have an air void content of about 4 or 5 percent (normal for Marshall procedure), the compacted specimen will have very low air permeability regardless of aggregate gradation. Therefore, the use of air permeability is not a necessary criterion for use in the design of dense graded asphalt mixtures.
- When aggregate gradation is a variable, none of the factors—air void content, air permeability, or bituminous film thickness expressed as bitumen index—can be used singly to satisfactorily indicate a mixture's resistance to asphalt hardening. Test results indicate that use of a combined factor of a ratio of air void content to bitumen index is satisfactory for comparing resistance to asphalt hardening of different mixtures, regardless of the gradation of the aggregate blend. Air permeability is not required as a part of the combined factor.
- The Marshall method of mix design might be improved by the substitution of a maximum voids-bitumen index ratio for the presently used maximum air void content. Confirmation of this conclusion could not be established because of the limitations of the study. However, further research on the possibility of this change is desirable.

Materials Used in Study

The mixtures used for the study were prepared from an 85 to 100 penetration grade of asphalt and five different combinations (blends) of aggregate—½-inch in maximum size and containing crushed stone, natural sand, and limestone dust. The selection of the blends used in this study was based on the BPR Gradation Chart, which will be discussed later. The properties of the asphalt and aggregates are given in tables 1 and 2. The gradations of the component and combined aggregates are listed in table 2, which also contains data on surface area and on apparent, effective, and bulk specific gravities.

The gradations of the five blends of aggregate are identified and illustrated on the BPR Gradation Chart of figure 1. Aggregate blend C, of which 47 percent passed the No.

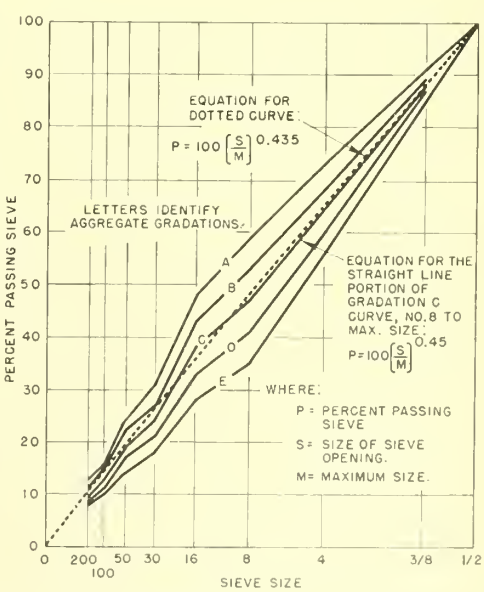

Figure 1.—Gradation of aggregate blends used.

8 sieve and 9.5 percent passed the No. 200 sieve, was intended to represent a maximum density blend. For the most part, blend C plots as a straight line from 0 percent passing the theoretical 0 sieve size to 100 percent passing the maximum sieve size. The aggregate gradations for blends B and A, respectively, were selected so that 6 and 12 percentage points more material would pass the No. 8 sieve than for blend C. Likewise, the aggregate gradations for blends D and E, respectively, were selected so that 6 and 12 percentage points less material would pass the No. 8 sieve than for blend C. The gradations of the parts of the aggregates that pass the No. 8 sieve, converted to 100 percent passing the sieve, were selected so that they would be approximately the same for all five blends.

Test Conducted

The test specimens were 4 inches in diameter and 2½ inches high and were molded by the gyratory procedure described under the heading *Details of Study*. Initially, specimens were made with different amounts of asphalt to determine the asphalt contents that would produce approximately 4-percent air void content for each of the five blends. These results were established as the highest asphalt contents to be used in the study. Intermediate and low asphalt contents were established by an arbitrary reduction of 1 and 2 percentage points, respectively, in the amount of asphalt. The intermediate and low asphalt contents produced about 7- and 10-percent air voids, respectively, in the compacted specimen.

Two series of tests were conducted. For the first series, mixtures were prepared with each of the five blends of aggregate at their respective three predetermined asphalt contents. Three specimens of each mixture were compacted for testing for each of four curing conditions: (1) no oven curing or outdoor storage; (2) 12-day oven curing at 140° F.; (3) 63-day oven curing at 140° F.; and (4) 303-day outdoor aging.

For the second series of tests, mixtures were prepared with three aggregate blends, A, C, and E, using each of the respective three predetermined asphalt contents. Four specimens of each mixture were compacted for each of the first two curing conditions. Specimens containing the three blends were also molded using the intermediate asphalt contents and tested for the third curing condition—63-day oven curing at 140° F. In each series of tests, the test specimens for each mixture were grouped so as to obtain sets comparable as to average density and time of room storage prior to start of curing. Details of procedure used in grouping are given in the discussion on details of the study.

Prior to their being grouped, all specimens were tested for bulk specific gravity, information from which mineral voids, air voids, and mineral voids filled with asphalt were computed. The average results for both series of tests are given in table 3, together with the asphalt contents used and the computed bitumen indexes.

When curing had been completed, the specimens were tested for Marshall stability and extracted for tests on recovered asphalt, which included penetration, softening point, and ductility. Before the Marshall tests were made, and a few times before they were cured, selected groups of specimens were tested for air permeability by three procedures, which will be described later.

Table 3.—Identification and physical characteristics of compacted specimens

Mixture			Test series 1,[1] compacted specimens				Test series 2,[2] compacted specimens			
Identification	Asphalt content by weight of aggregate	Bitumen index [3] (multiply by 10^{-3})	Bulk specific gravity	Mineral voids [4]	Air voids [5]	Mineral voids filled with asphalt	Bulk specific gravity	Mineral voids [4]	Air voids [5]	Mineral voids filled with asphalt
	Percent			*Percent*	*Percent*	*Percent*		*Percent*	*Percent*	*Percent*
AH	5.16	1.02	2.399	14.5	4.4	69	2.398	14.6	4.4	69
AM	4.16	.82	2.353	15.4	7.6	51	2.360	15.1	7.3	52
AL	3.16	.63	2.318	15.8	10.3	35	2.320	15.7	10.2	35
BH	4.88	1.05	2.415	14.0	4.5	68	-------	------	------	------
BM	3.88	.84	2.368	14.8	7.7	48	-------	------	------	------
BL	2.88	.62	2.332	15.3	10.4	32	-------	------	------	------
CH	5.03	1.25	2.427	13.9	4.2	70	2.428,	13.9	4.1	70
CM	4.03	1.00	2.380	14.8	7.4	50	2.381	14.7	7.3	50
CL	3.03	.75	2.342	15.3	10.2	34	2.342	15.3	10.2	34
DH	5.08	1.42	2.435	14.0	4.2	70	-------	------	------	------
DM	4.08	1.14	2.390	14.7	7.3	51	-------	------	------	------
DL	3.08	.86	2.348	15.4	10.2	34	-------	------	------	------
EH	5.35	1.70	2.435	14.5	4.2	71	2.439	14.3	4.0	72
EM	4.35	1.38	2.394	15.1	7.1	53	2.398	15.0	7.0	53
EL	3.35	1.06	2.346	16.0	10.3	35	2.355	15.7	10.0	36

[1] Average of 12 specimens for each mixture.
[2] Average of 12 specimens for AM, CM, and EM; and 8 specimens for each of other mixtures.
[3] Pounds of asphalt per ft.^{2} surface area of aggregate determined by California procedure.
[4] Based on bulk specific gravity of aggregate.
[5] Based on effective specific gravity of aggregate.

Exponent Used for BPR Gradation Chart

The exponent 0.45 used in establishing the horizontal scale of the BPR Gradation Chart was based on the findings of L. W. Nijboer and the research study on gravel mixtures by Goode and Lufsey described in a 1962 AAPT paper (*12*). Goode and Lufsey's research study suggested that 0.435 might be a better exponent but the authors agreed that it was not significantly different from 0.45, the exponent based on the earlier work of Nijboer; therefore, they decided to use the latter as a basis for the gradation chart.

Figure 1, which shows the aggregate gradations of the blends used in the study reported here on the BPR Gradation Chart, includes a dotted flat curve indicating a maximum density gradation based on the use of 0.435 as the exponent. This curve would have been a straight line had the entire figure been based on this exponent rather than on 0.45. Aggregate blend C was intended to represent maximum density. Its plotted gradation was for the most part a straight line connecting the point at 100 percent passing the maximum size and the point at 0 percent passing the theoretical 0 sieve size.

Aggregate blends B and D were intended to have appreciably lower densities, but the mineral voids curves of figure 2 show that they did not. Blend B was as dense as blend C, and D was almost as dense. Therefore, the best equation for obtaining data on maximum density appears to be one that can be plotted between the curves for blends C and B but closer to the curve for blend C and one having 0.435 as the exponent—such as is shown by the dotted curve in figure 1. Thus, there is an indication confirming that 0.435 might be a better exponent than 0.45 for determining maximum density for mixtures containing ½-inch maximum size crushed stone. However, it is doubtful that the slight difference in the results obtained by use of the two exponents is sufficient to justify revision of the BPR Gradation Chart.

Details for Air Permeability Tests

Figures 3 and 4 are, respectively, a photograph and a schematic drawing of the air permeability apparatus used in the study reported here. The apparatus and the method of its use differ from that developed by Ellis and Schmidt (*8*) in that airflow is created by use of a vacuum of an air pressure line and the rate of airflow is determined by use of one of three different size flowmeters. Calibration

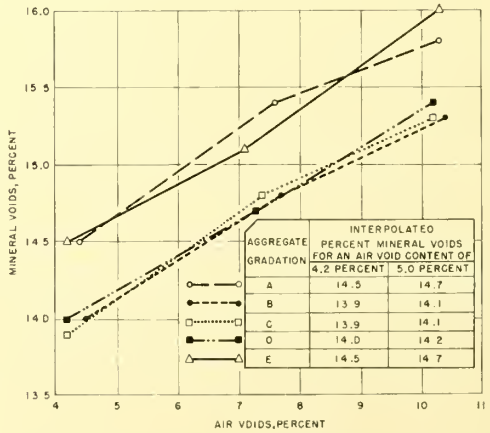

Figure 2.—Relation between air voids and mineral voids for specimens of test series 1.

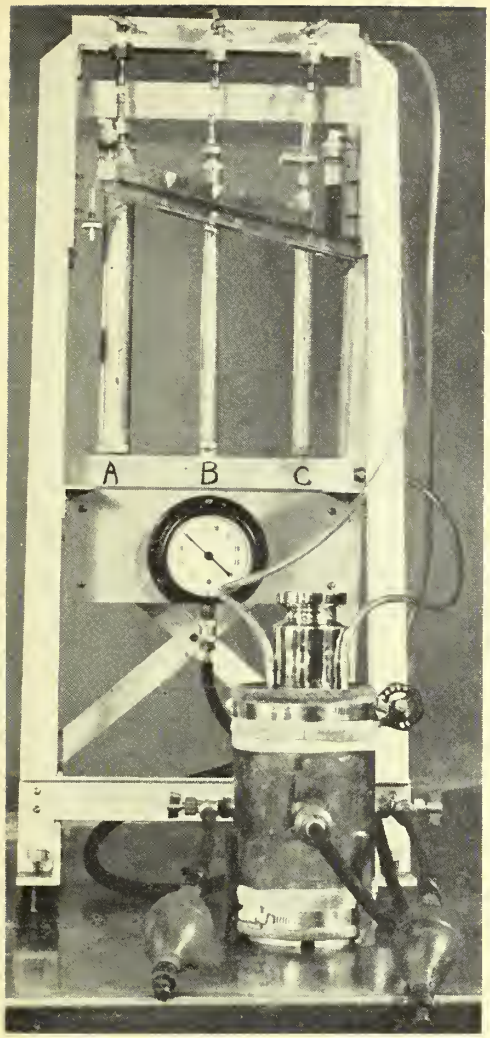

Figure 3.—Apparatus used to measure air permeability.

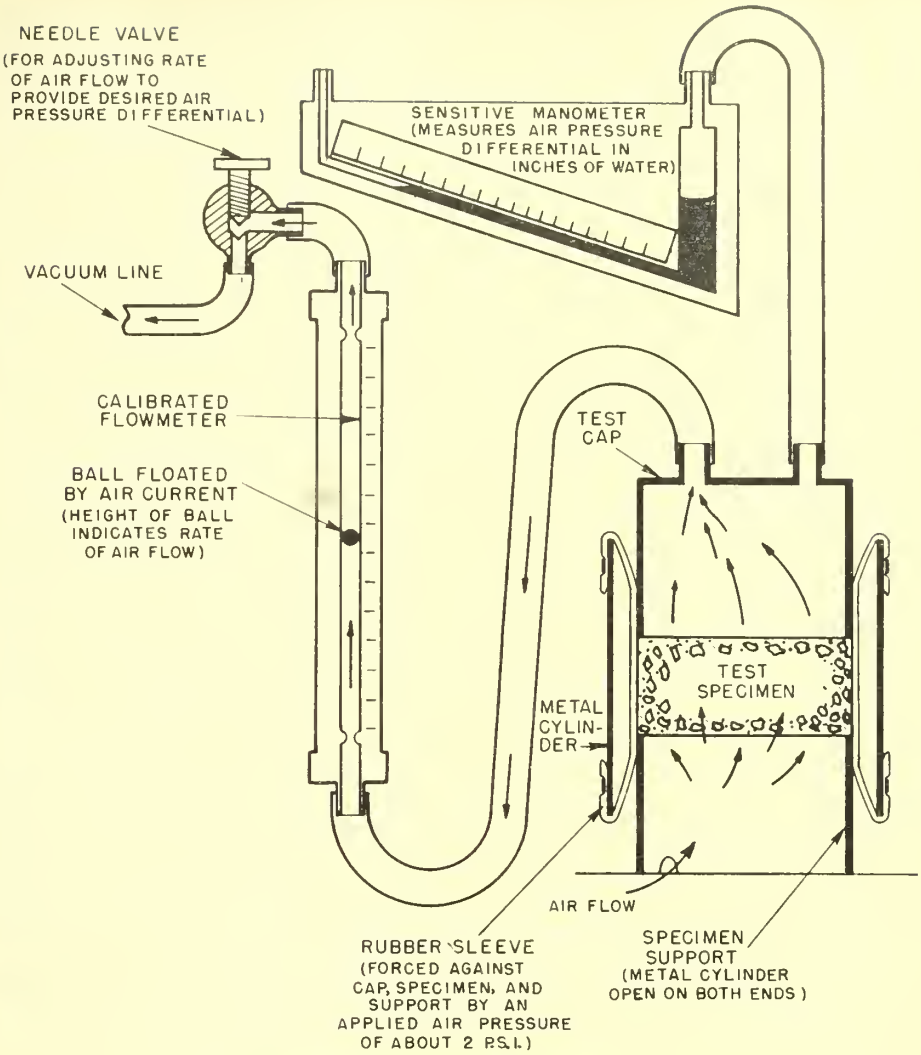
NEEDLE VALVE
(FOR ADJUSTING RATE OF AIR FLOW TO PROVIDE DESIRED AIR PRESSURE DIFFERENTIAL)
SENSITIVE MANOMETER (MEASURES AIR PRESSURE DIFFERENTIAL IN INCHES OF WATER)
VACUUM LINE
CALIBRATED FLOWMETER
BALL FLOATED BY AIR CURRENT (HEIGHT OF BALL INDICATES RATE OF AIR FLOW)
TEST CAP
TEST SPECIMEN
METAL CYLINDER
AIR FLOW
RUBBER SLEEVE (FORCED AGAINST CAP, SPECIMEN, AND SUPPORT BY AN APPLIED AIR PRESSURE OF ABOUT 2 P.S.I.)
SPECIMEN SUPPORT (METAL CYLINDER OPEN ON BOTH ENDS)

Figure 4.—Air permeability test.

charts are used for converting flowmeter readings to rates of airflow in millimeters per minute. The principal advantage of using this apparatus over others employing a falling head of water for creating airflow is that timing of flow is not required while maintaining a consistent pressure differential.

Preliminary work with the Public Roads apparatus showed that to obtain reliable measurements of permeability, a seal on the vertical face of the test specimen was required most of the time to prevent leakage of air between the side of the specimen and the rubber membrane. Paraffin, applied hot with a brush, was an effective sealing material.

Paraffin was used as a seal for the uncured and oven cured specimens of the first series of tests. Before the stability tests were made the specimens were warmed slightly in an oven and most of the paraffin was removed. In an effort to prevent paraffin contamination, the outer one-fourth of an inch of the specimen was discarded before the extraction. Unfortunately, as discovered later, sufficient paraffin contamination remained to make the results of tests on these extracted asphalts useless. Therefore, the second series of tests was conducted. Wet clay was tried as a seal on several of the remaining specimens, and a limited amount of data was obtained from specimens on which no seal had been used.

Tables 4, 5, and 6 contain the permeability results for the tests made when the vertical faces of the specimens were sealed with paraffin, with wet clay, and unsealed, respectively. Rates of airflow were determined for pressure differentials of ½, 1, and 2 inches of water. Permeability in fundamental units was computed, as described in the details of the study, and averaged.

Table 7 contains a comparison of the effects of no seal, the paraffin seal, and the wet clay seal on the determined permeability for three different aggregate blends and at three levels of air void content. The specimens sealed with wet clay had less permeability than those sealed by paraffin, although at the high air void content of 10 percent the percentage differences between permeability were small. When wet clay was used, indications that moisture had entered the voids of the specimens were noted for several specimens during the testing. Therefore, it is believed that the results indicating lower permeability were caused by moisture blocking some of the capillaries of the specimen and not that wet clay was a more effective seal than paraffin.

All the specimens not sealed had higher permeabilities than those sealed with the paraffin. The percentage differences in permeability were slight at 10-percent air voids but very pronounced at 4-percent voids, especially for blend E, the coarsest of the three.

Results of the tests, which are given in table 7, show that the vertical face of the specimen requires sealing for reliable values of permeability. Paraffin is an effective sealing material but should not be used when the specimen is to be extracted for tests on the recovered asphalt. There probably are other sealing materials that would be suitable for extraction tests as well as permeability tests, but this was not investigated. The particular wet clay used in the study reported here did not appear to be a suitable sealing agent because of the possible blocking of airflow channels by moisture.

Effect of Gradation and Air Voids on Permeability

The relationships obtained between air void content, gradation of aggregate blend, and air permeability when paraffin was used

Table 4.—Air permeability results when paraffin was used to seal vertical face of specimen

Mix identification	Rate of airflow [1] for specimens from test series 1, for pressure differential of—									Computed average permeability [2] for pressure differential of—			
	½ inch of water			1 inch of water			2 inches of water			½ inch of water	1 inch of water	2 inches of water	Average
	N-1 [3]	P-1 [4]	Average	N-1 [3]	P-1 [3]	Average	N-1 [3]	P-1 [4]	Average				
	Ml./min.	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*K*	*K*	*K*	*K*
AH	2	2	2	6	5	5.5	15	12	13.5	4	5.5	6.5	5.5
AM	27	29	28	54	53	54	101	108	104	55	53	51	53
AL	53	52	52	108	105	106	213	207	210	101	103	102	102
BH	3	3	3	7	6	6.5	16	13	14.5	6	6.5	7	6.5
BM	35	38	36	67	73	70	129	143	136	70	68	66	68
BL	72	68	70	143	140	142	278	270	274	136	138	133	136
CH	3	3	3	7	5	6	21	10	16	6	6	8	6.5
CM	45	42	44	86	78	82	171	157	164	86	80	80	82
CL	125	127	126	238	248	243	502	515	508	245	237	247	243
DH	5	3	4	9	4	6.5	25	8	16.5	8	6.5	8	7.5
DM	59	57	58	111	113	112	215	222	218	113	109	106	109
DL	217	217	217	455	447	451	903	890	896	423	439	436	433
EH	7	2	4.5	14	4	9	31	5	18	9	9	9	9
EM	72	63	68	139	113	126	240	203	222	132	123	108	121
EL	571	525	548	1,033	919	976	1,812	1,719	1,766	1,068	951	860	960

[1] A 2-p.s.i. pressure was used to force rubber membrane against side of specimen. Vacuum was used to produce airflow.
[2] Permeability per cm.², cm.³ per sec., K (multiply by 10^{-10}).
[3] Uncured group of specimens.
Oven cured group of specimens, after 12 days at 140° F.

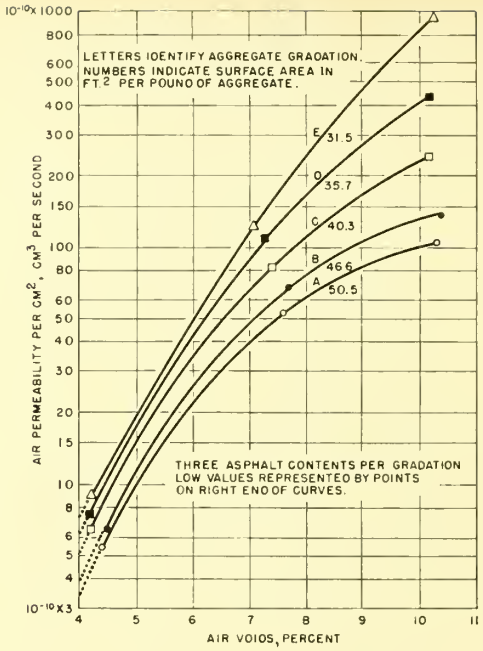

Figure 5.—Relation between air voids and permeability.

as a seal are shown in figure 5. The different contents of air voids were obtained by the use of different asphalt contents rather than different compactive efforts. All specimens were compacted by a gyratory compaction procedure that produced a density close to what would be obtained from the 50-blow Marshall procedure.

In confirmation of the findings of Hein and Schmidt (*9*), data in figure 5 show very definitely that permeability is not necessarily proportional to air voids when aggregate gradation is a variable. As noted previously, the order of aggregate gradation of the blends in sequence was from finest to coarsest A, B, C, D, and E. The magnitude of permeability at all void contents followed the same sequence (fig. 5). For example, at 7-percent air voids the permeabilities for the finest to the coarsest gradation were, respectively 40, 48, 65, 90, and 115 each $\times 10^{-10}$ cm.³ per sec.

The effect of aggregate gradation was much more pronounced when air void content was high than when low; for example, at 10-percent air void content, specimens made from aggregate blend E had a permeability of 800×10^{-10} cm.³ per sec., which was 8 times that for the specimens containing aggregate A; at 4-percent air void content, the specimens made with aggregate blend E had a very low permeability, 7×10^{-10} cm.³ per sec., which was only twice that for specimens containing aggregate blend A. Because of the low permeabilities that might be expected when air void content is 4 or 5 percent, regardless of aggregate gradation, no need is apparent for an air permeability requirement in a mix design procedure for a dense-graded mixture in which the asphalt content is established to produce about 4 percent of air voids.

Bitumen Index

Bitumen index as determined by the California procedure was selected to represent film thickness. An index was employed to avoid an implication that all particles are coated with a uniform thickness of bituminous material. If it is desirable to convert bitumen index to film thickness in microns, as employed by Campen and others (*5*), this can be done by multiplying the bitumen index by 4,870. These authors indicated that the film thickness should be at least 6 microns to obtain satisfactory pavement performance. This corresponds to a bitumen index of at least 1.23×10^{-3}. From test data presented

Table 5.—Air permeability when wet clay was used to seal vertical face of specimen

Mix identification [1]	Test series 1, specimens [2] from group S-1													Test series 2, specimens [3] from group N-2								
	Airflow rate [4] for pressure differential of—									Air voids	Computed permeability [5]				Airflow rate [4] for pressure differential of—			Air voids	Computed permeability [5]			
	½ inch of water			1 inch of water			2 inches of water				For pressure differential of—			Average	½ inch of water	1 inch of water	2 inches of water		For pressure differential of—			Average
	Prior to storage	After storage	Average	Prior to storage	After storage	Average	Prior to storage	After storage	Average		½ inch of water	1 inch of water	2 inches of water						½ inch of water	1 inch of water	2 inches of water	
	Ml./min.	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Percent*	*K*	*K*	*K*	*K*	*Ml./min.*	*Ml./min.*	*Ml./min.*	*Percent*	*K*	*K*	*K*	*K*
AM	15	12	14	30	22	26	59	42	50	7.6	27	25	24	25	10	22	43	7.3	19	21	21	20
AL	42	46	44	80	94	87	159	192	176	10.3	86	85	86	86	42	84	171	10.2	82	82	83	82
CM	14	23	18	27	44	36	52	84	68	7.4	35	35	33	34	34	63	123	7.3	66	61	60	62
CL	107	89	98	200	183	192	399	378	388	10.2	191	187	189	189	105	204	409	10.2	205	199	199	201
EM	13	----	13	19	-----	19	33	-----	33	7.1	25	19	16	20	32	58	100	7.0	62	56	49	56
EL	430	428	429	741	781	761	1,329	1,315	1,322	10.3	836	741	644	740	541	910	1,589	10.0	1,054	886	774	905

[1] Specimens for mixtures AH, CH, and EH were tested also. Their rates of airflow were less than 2 ml. per minute, the lowest rate that can be measured with the apparatus, and permeabilities were less than 1×10^{-10}.
[2] Specimens from outdoor storage group.
[3] Uncured group of specimens.
[4] A 3-p.s.i. pressure was used to force rubber membrane against vertical face of specimen. Pressure was used to produce airflow for S-1 specimens and vacuum for N-2 specimens.
[5] Air permeability per cm.², cm.³ per sec., K (multiply by 10^{-10}).

Table 6.—Air permeability when no seal was used for vertical face of specimen [1]

Mix identification	Rate of airflow,[2] for pressure differential of 1 inch of water	Computed permeability [3]
	Ml. per min.	*K*
AH	22	21
AM	61	59
AL	130	127
CH	119	116
CM	169	165
CL	302	294
EH	436	425
EM	676	658
EL	1,547	1,507

[1] Test series 2, specimens from group P-2 oven cured 12 days at 140° F.
[2] A 3-p.s.i. air pressure was used to force rubber membrane against vertical face of specimen. Pressure was used to produce airflow.
[3] Permeability per cm.², cm.³ per sec., *K* (multiply by 10^{-10}).

Table 7.—Effect on measured permeability when the vertical face of specimen was not sealed with paraffin, or sealed with clay

Air voids for aggregate blends	Interpolated permeability [1]		
	Not sealed	Sealed with paraffin [2]	Sealed with wet clay [3]
Percent	*K*	*K*	*K*
A:			
4.0	18	3	<1
7.0	54	40	19
10.0	122	98	75
C:			
4.0	114	5	<1
7.0	160	67	40
10.0	280	230	175
E:			
4.0	425	7	<1
7.0	658	115	37
10.0	1,507	800	710

[1] From data of tables 3 through 6. Permeability per cm.², cm.³ per sec., *K* (multiply by 10^{-10}).
[2] Tests had shown paraffin to be an effective seal.
[3] Wet clay appears to be effective in sealing vertical face of specimen, but it is indicated that moisture from the wet clay enters and tends to seal capillaries of the specimen, thereby causing decrease in permeability.

Table 8.—Surface area of aggregate and bitumen index compared with film thickness

Aggregate		Mixture [1]			
Blend	Surface area, ft.² per lb. of aggregate	Identification	Asphalt content,[2] percent	Bitumen index	Film thickness, microns
A	50.5	AH	5.16	1.02×10^{-3}	[3] 5.0
B	46.4	BH	4.88	1.05×10^{-3}	[3] 5.1
C	40.3	CH	5.03	1.25×10^{-3}	6.1
D	35.7	DH	5.08	1.42×10^{-3}	6.9
E	31.5	EH	5.35	1.70×10^{-3}	8.3

[1] At asphalt content producing about 4-percent air voids in compacted specimens.
[2] Aggregate basis.
[3] Film thickness less than 6 microns.

in table 8, a comparison can be made of the bitumen index and film thickness for each of the five aggregate blends at which the asphalt contents produced about 4-percent air void content in the specimens. Aggregate blends A and B would be considered unsatisfactory by the 6-micron thickness criterion. These two blends would normally be considered unsuitable for asphaltic concrete surface course mixtures because of their high dust contents, 12.3 and 11.3 percent, respectively. When blend C, having the most dense gradation of blends, was used for the test specimens, the film was approximately at the 6-micron thickness criterion.

Effect of Mix Properties on Asphalt Hardening

The data used in studying the effect of mix properties on asphalt hardening are summarized in tables 9 and 10. From the data for the uncured and cured specimens, three methods for rating asphalt hardening are suggested: (1) percentage of retained penetration, (2) percentage gain in Marshall stability, and (3) change in softening point temperature. The latter method proved to be less suitable than the other two and was not used. The first two methods were used in determining the data for figures 6, 7, 8, and 9 to show the effect of the variables on the degree of asphalt hardening in the molded specimens subjected to 12 days of oven curing at 140° F. These figures were developed from the data obtained from the second series of tests in which only three aggregate blends were used, A, C, and E.

The relationships between bitumen index per se and degree of asphalt hardening for each of the three blends at three asphalt contents are shown in figure 6. The asphalt contents are not identified on the figure but the higher asphalt contents are represented by the points on the right end of the curves. In the upper part of the figure, retained penetration indicates resistance to hardening and in the lower part, gain in Marshall stability.

The curves at the top or bottom of figure 6 show, for a particular blend, a definite trend for asphalt hardening to decrease as bitumen index increases; the retained penetration increased and the gain in Marshall stability decreased. These data indicate but do not prove that bitumen index is an important factor affecting asphalt hardening. The fact that the curves for the three aggregate blends are widely separated indicates that at least one other factor also affects the rate of asphalt hardening.

In figure 7 the retained penetration and gain in Marshall stability have been plotted against air permeability instead of the bitumen index (fig. 6). Data for the specimens having high asphalt content are on the left. The curves in figure 7 illustrate the definite trend, for a particular aggregate blend, which indicates that asphalt hardening increases as permeability increases. The wide spread between the curves again indicates that at least one other factor affects the rate of asphalt hardening.

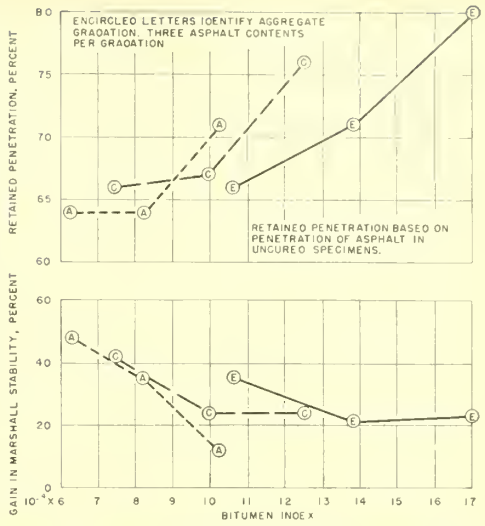
ENCIRCLED LETTERS IDENTIFY AGGREGATE GRADATION. THREE ASPHALT CONTENTS PER GRADATION
RETAINED PENETRATION, PERCENT
RETAINED PENETRATION BASED ON PENETRATION OF ASPHALT IN UNCURED SPECIMENS.
GAIN IN MARSHALL STABILITY, PERCENT
BITUMEN INDEX

Figure 6.—Effect of bitumen index on asphalt hardening after 12 days of oven curing at 140° F.

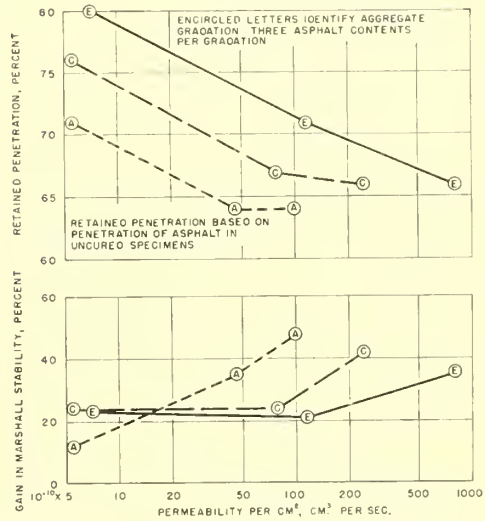
ENCIRCLED LETTERS IDENTIFY AGGREGATE GRADATION THREE ASPHALT CONTENTS PER GRADATION
RETAINED PENETRATION, PERCENT
RETAINED PENETRATION BASED ON PENETRATION OF ASPHALT IN UNCURED SPECIMENS
GAIN IN MARSHALL STABILITY, PERCENT
PERMEABILITY PER CM², CM³ PER SEC.

Figure 7.—Effect of permeability on asphalt hardening after 12 days of oven curing at 140° F.

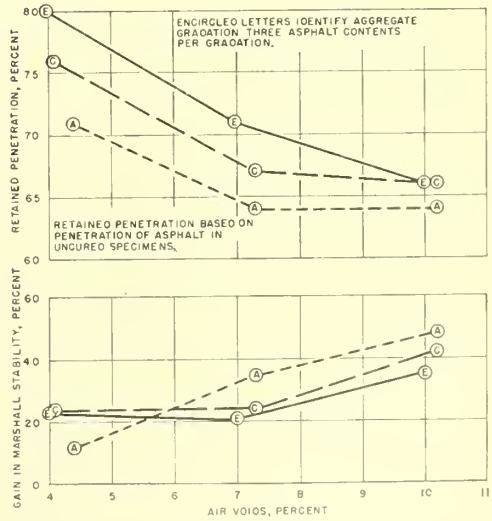
ENCIRCLED LETTERS IDENTIFY AGGREGATE GRADATION THREE ASPHALT CONTENTS PER GRADATION.
RETAINED PENETRATION, PERCENT
RETAINED PENETRATION BASED ON PENETRATION OF ASPHALT IN UNCURED SPECIMENS.
GAIN IN MARSHALL STABILITY, PERCENT
AIR VOIDS, PERCENT

Figure 8.—Effect of air voids on asphalt hardening after 12 days of oven curing at 140° F.

Table 9.—Results of test on specimens aged outdoors, test series 1

Mix identification	Air voids	Bitumen index (multiply by 10^{-3})	Voids-bitumen index ratio [1]	Uncured specimens [2]			Specimens aged outdoors 303 days [5]				
				Permeability [3]	Marshall [4]		Marshall [4]			Recovered asphalt [5]	
					Stability	Flow	Stability	Flow	Increase in stability	Penetration	Softening point
	Percent			*K*	*Pounds*		*Pounds*		*Percent*		° *F.*
AH	4.4	1.02	4.3	5.5	1,750	8	2,510	12	43	40	131
BH	4.5	1.05	4.3	6.5	1,770	9	2,530	11	43	40	132
CH	4.2	1.25	3.4	6.5	1,650	10	2,160	11	31	------	132
DH	4.2	1.42	3.0	7.5	1,580	11	2,010	12	27	40	131
EH	4.2	1.70	2.5	9	1,300	13	1,700	14	31	44	128
AM	7.6	.82	9.3	53	1,470	8	2,200	11	50	38	132
BM	7.7	.84	9.2	68	1,400	8	2,250	11	61	40	130
CM	7.4	1.00	7.4	82	1,300	8	2,180	11	68	39	------
DM	7.3	1.14	6.4	109	1,300	8	1,920	11	48	38	130
EM	7.1	1.38	5.1	121	1,230	9	1,760	12	43	39	131
AL	10.3	.63	16.3	102	870	7	1,590	10	83	38	129
BL	10.4	.62	16.8	136	960	6	1,540	10	60	38	129
CL	10.2	.75	13.6	243	1,060	7	1,680	9	58	38	131
DL	10.2	.86	11.9	433	1,100	6	1,680	10	53	37	132
EL	10.3	1.06	9.7	960	940	7	1,690	11	80	37	132

[1] Percent air voids divided by (10^3 × bitumen index).
[2] Precoating with paraffin contaminated the asphalt and the results of tests on recovered asphalt were discarded. Marshall stability also could have been affected.
[3] Permeability per cm.^{2}, cm.^{3} per sec., K (multiply by 10^{-10}).
[4] Averages of three tests.
[5] Only one recovery for each mixture, average ductility was at least 230 cm.

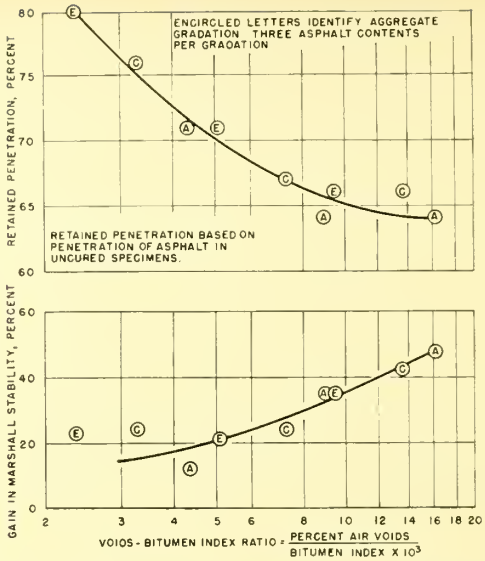
ENCIRCLED LETTERS IDENTIFY AGGREGATE GRADATION. THREE ASPHALT CONTENTS PER GRADATION
RETAINED PENETRATION, PERCENT
RETAINED PENETRATION BASED ON PENETRATION OF ASPHALT IN UNCURED SPECIMENS.
GAIN IN MARSHALL STABILITY, PERCENT
VOIDS - BITUMEN INDEX RATIO = PERCENT AIR VOIDS / BITUMEN INDEX X 10³

Figure 9.—Effect of voids-bitumen index ratio on asphalt hardening after 12 days of oven curing at 140° F.

The effect of air voids per se on the degree of asphalt hardening for each of the three aggregate blends is illustrated in figure 8. The specimens having high asphalt content are represented by the points to the left. The curves in the figure show that the effect of air voids on hardening was very similar to the effect on permeability. These data also indicate that at least one other factor affects the rate of asphalt hardening.

When aggregate gradation is a variable, as shown in figures 6, 7, and 8, none of the factors alone—bitumen index, permeability, or air void content—can be used to properly evaluate a compacted mixture as to its resistance to hardening of the asphalt during service. The results of the tests indicated that a combination of two or of all three factors was needed for a satisfactory evaluation. Several combined factors were tried. The one that appears to be most suitable on the basis of the data of the study consists of a simple ratio of the percentage of air void content divided by the product of the bitumen index × 10^3.

Voids-bitumen index ratios for the mixtures containing the three blends of aggregate used in test series No. 2 are given in table 10. These ratios were plotted against the retained penetration and gain in Marshall stability, as shown in figure 9. The use of the ratio, which combines the void content and bitumen index factors, produces a single curve for all blends. The maximum deviations from the plotted curves were only 2 percentage points for retained penetration, and about 10 percentage points for gain in Marshall stability. These deviations are considered within or reasonably close to the precision of the tests. For the penetration test, accepted results of repeatability and reproducibility are 4- and 8-percent, respectively. Attempts were made to include permeability in a combined factor but no logical formula was found that would provide a better fit of the plotted data than that shown in figure 9.

In figure 10 the voids-bitumen index ratio is used to compare asphalt hardening after 303 days of outdoor exposure for the 15 mixtures of the first series of tests. As mentioned previously, it had become necessary to discard the results of tests on the asphalt recovered from the uncured specimens because of the paraffin contamination of the specimens. The

Table 10.—Results of tests on oven cured specimens, test series 2

Mix identification	Air voids	Bitumen index (multiply by 10^{-3})	Voids-bitumen index ratio [1]	Permeability [2]	Curing at 140° F.	Marshall [3]			Recovered asphalt [4]		
						Stability	Flow	Increase in stability	Penetration	Retained penetration	Softening point
	Percent			*K*	*Days*	*Pounds*		*Percent*		*Percent*	° *F.*
AH	4.4	1.02	4.3	5.5	0	2,200	10	------	66	------	120
					12	2,470	10	12	47	71	127
CH	4.1	1.25	3.3	5.5	0	1,920	10	------	66	------	120
					12	2,380	10	24	50	76	128
EH	4.0	1.70	2.4	7	0	1,440	10	------	71	------	120
					12	1,770	11	23	57	80	123
AM	7.3	.82	8.9	46	0	1,680	8	------	64	------	121
					12	2,270	8	35	41	64	130
					63	2,540	10	51	32	50	136
CM	7.3	1.00	7.3	78	0	1,570	8	------	66	------	121
					12	1,950	9	24	44	67	130
					63	2,450	11	56	31	47	136
EM	7.0	1.38	5.1	115	0	1,460	9	------	66	------	121
					12	1,760	10	21	47	71	127
					63	2,160	12	48	34	52	135
AL	10.2	.63	16.2	100	0	1,110	8	------	64	------	121
					12	1,640	9	48	41	64	129
CL	10.2	.75	13.6	243	0	1,100	6	------	64	------	120
					12	1,560	9	42	42	66	131
EL	10.0	1.06	9.4	800	0	1,180	8	------	62	------	121
					12	1,590	8	35	41	66	127

[1] Percent air voids divided by (10^3 × bitumen index).
[2] Permeability per cm.^{2}, cm.^{3} per sec., K (multiply by 10^{-10}).
[3] Averages of four tests.
[4] Averages of tests from two recoveries; average ductility was at least 210 cm.

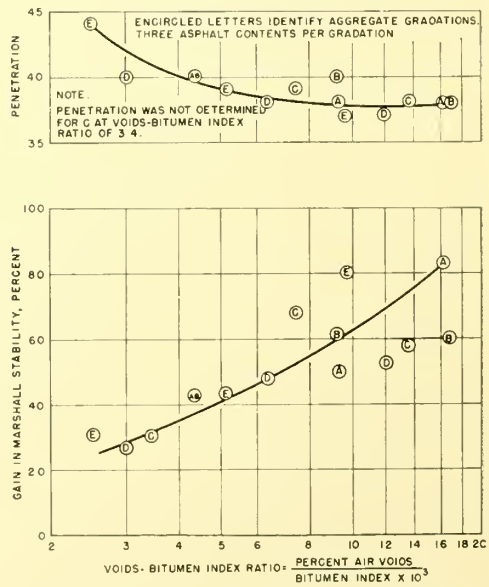
ENCIRCLED LETTERS IDENTIFY AGGREGATE GRADATIONS. THREE ASPHALT CONTENTS PER GRADATION
PENETRATION
NOTE: PENETRATION WAS NOT DETERMINED FOR C AT VOIDS-BITUMEN INDEX RATIO OF 3.4.
GAIN IN MARSHALL STABILITY, PERCENT
VOIDS - BITUMEN INDEX RATIO = PERCENT AIR VOIDS / BITUMEN INDEX X 10³

Figure 10.—Effect of voids-bitumen index ratio on asphalt hardening after 303 days of outdoor storage.

asphalt recovered from specimens stored outdoors was not contaminated as they had not been coated with paraffin. Because suitable data were unavailable for computing retained penetration, the actual penetration was used in the plotting for the upper part of figure 10; as in the other figures, gain in Marshall stability was used to plot the lower part.

Penetration data in the upper part of figure 10, show somewhat more dispersion from the plotted curve than for the oven cured specimens of the second series of tests, as shown in figure 9. However, for this second series of tests, two extractions and recoveries were made for each curing period compared to one recovery for the outdoors weathered specimens of the first series of tests. In the upper part of figure 10, penetrations for two of the mixtures—the ones indicated by the first D and second B from the left side—are shown as 2 to 3 points from the plotted curve, but these penetrations are within the reproducibility of the penetration test method. If these 2 points were disregarded, the plotted points would be considered in excellent agreement with the curve and this would indicate that the voids-bitumen index ratio is a good measure for evaluating the resistance to asphalt hardening of a compacted mixture.

Information in the lower part of figure 10 also shows somewhat more dispersion of gain in Marshall stability from the plotted curve than the data shown in figure 9. The greater dispersion could be attributed to the operational factors associated with the long curing procedure. Nevertheless, these data serve to substantiate the fact that the voids-bitumen index ratio is a useful tool for evaluating the effect of aging on the stability of paving mixtures.

Voids-Bitumen Index Ratio in Mix Design

In the discussion of data in table 8, aggregate blends A and B were mentioned as having too high a dust content for satisfactory use in bituminous paving mixtures. At design asphalt contents, for both aggregates the bitumen index was less than 1.23×10^{-3} the minimum for satisfactory pavement performance based on the work of Campen and others (*5*). This minimum bitumen index could very well be included as a criterion in all mix design procedures.

A substitute and possibly a better criterion for the 50-blow Marshall procedure might be the use of an upper limit on the voids-bitumen index ratio and the elimination of the upper limit on percentage of air voids. From data in figure 10 and table 9, a criterion of a maximum of 4.0 for the voids-bitumen index ratio would eliminate the use of aggregate blends such as A and B at all asphalt contents used in the study reported here and would permit the use of the other three blends only at the higher asphalt contents. The degree of asphalt hardening was definitely less severe when the voids-bitumen index ratios were 4 or less (figs. 9 and 10).

The use of a minimum air void content of 3 percent and a maximum voids-bitumen index ratio of 4 in a mix design procedure and the compaction method used for the study discussed here—in which the density approximates the density of a 50-blow Marshall specimen—would permit differences in air void content according to the aggregate blend used. For example, the maximum air void contents that would be permitted for blends C, D, and E were interpolated from plotted curves of data from table 9 as 4.7, 5.1, and 6.0 percent, respectively. Data obtained from other plotted curves showed that the differences in air void content, from 3.0 percent to these maximum air void contents, corresponded to differences between maximum and minimum asphalt contents of 0.6, 0.7, and 1.1 percentage points, respectively. Thus, a wider range in asphalt content could be used with aggregate blend E than blends D or C. This seems logical as aggregate blend E was coarser and more open in gradation.

DETAILS OF STUDY

Preliminary Tests

Tables 1 and 2 list the properties of the asphalt and aggregates used in the study. Prior to the preparation of test specimens the crushed stone was separated into ½-inch to ⅜-inch, ⅜-inch to No. 4, and No. 4 to No. 8 sizes. Each of the three sands was thoroughly mixed but not sieved into smaller sizes. The cumulative system of weighing, starting with coarsest materials, was used in preparing the batches of aggregate for the study. Each batch was sufficient for only one test specimen.

Enough preliminary mixtures and test specimens were prepared and tested to establish, for each of the five aggregate blends: (1) effective specific gravity of the aggregate; (2) asphalt contents that would produce about 4-percent air void content in the compacted specimen; and (3) batch weights required for each mixture to produce the proper size of test specimen, 4 inches in diameter and 2½ inches high.

Preparation of Mixtures and Specimens

The mixtures were prepared in a laboratory mixer from aggregate that had been heated to 325° F. and asphalt that had been heated to 300° F. The individual batches of aggregate were heated overnight in an oven. A gyratory compactor, of the type developed by the U.S. Army, Corps of Engineers, was used to mold the test specimens. Immediately after its preparation, each mixture was placed in a gyratory mold, heated to 200° F., and compacted by application of 30 gyrations at a 1° angle while it was under a foot pressure of 100 p.s.i. The same compactor and molding procedures were used in an earlier study (*12*) on aggregate gradations.

First Series of Tests

Specimen grouping and testing

Fifteen different mixtures were used in the first series of tests: Each of the 5 aggregate blends was used in mixtures having high, medium, and low asphalt contents. A total of 180 specimens was molded to provide 3 specimens from each mixture and for each of 4 curing conditions.

The specimens having a high asphalt content, medium asphalt content, and low asphalt content were prepared and processed at intervals of about 2 weeks. The procedure used was as follows:

Four specimens from each of the 5 mixtures were molded on each of 3 consecutive days. Bulk specific gravities, determined by the procedure described in paragraph 4 of AASHO Designation: T 165–55 (*Effect of Water on Cohesion of Compacted Bituminous Mixtures*) were averaged for the specimens of each mixture. The 12 specimens of each mixture were then sorted into 4 groups so that each group of 3 contained a specimen from each day of molding and had approximately the same average bulk specific gravity. The 4 test groups of specimens were identified as:

N, No curing other than storage in laboratory air.

P, Storage in laboratory air plus 12 days in an oven at 140° F.

Q, Storage in laboratory air plus 63 days in an oven at 140° F.

S, Outdoor aging.

The *N* groups of specimens were tested for air permeability a few days after being molded. Paraffin was applied to their vertical faces before the permeability tests were made. On the ninth day after the first day of molding and after removal of paraffin, the *N* groups of specimens were tested for Marshall stability. These specimens were then sealed in cans so that later they could be extracted to determine the properties of the recovered asphalt. Upon completion of the oven curing, the specimens from the *P* and *Q* groups were tested for permeability and stability and sealed in cans in the same manner as the *N* groups.

The *S* groups of specimens were tested for permeability by using wet clay to seal their vertical faces. The specimens were then washed clean and air dried. Prior to their outdoor storage, aluminum foil was wrapped around the side of each specimen and folded down to completely cover the top surface. The wrapped specimens were then placed on a 1-inch bed of sand in a wooden frame that was on the flat roof of a concrete building. Sand was added to fill in the space between the specimens and was sealed on the surface by an application of a diluted asphalt emulsion. After the emulsion had set, the aluminum foil covering the tops of the specimens was removed. The specimens were stored outdoors for 303 days.

Second Series of Tests

Specimen grouping and testing

Nine different mixtures were used in the second series of tests. Three different aggregate blends were used in mixtures having high, medium, and low asphalt contents. A total of 84 specimens was molded from these 9 mixtures: 4 from each mixture for each test condition. Specimens from each mixture

were cured by the procedures N and P, used in the first series of tests, and method Q was used to cure only specimens from mixtures having medium asphalt contents. On each of 4 consecutive molding days, 2 specimens were prepared from each of the 6 high- and low-asphalt content mixtures, and 3 specimens were prepared from each of the 3 medium-asphalt content mixtures. Bulk specific gravities were determined for each of the specimens and all were averaged for each mixture. The specimens from each mixture were sorted into groups of 4 specimens so that each group contained a specimen from each day of molding and all groups had approximately the same average bulk specific gravity.

The 4-specimen test groups for the second series of tests were processed the same as the 3-specimen test groups in the first series of tests except that: (1) The vertical faces of the specimens were not coated with paraffin prior to permeability tests. (2) Upon completion of Marshall stability tests, the specimens and small amounts of steel wool, were placed in cans of distilled water and subjected to a vacuum of 27 inches of mercury for about 30 minutes. Earlier work had indicated that this was an effective means of preventing excessive hardening of the asphalt while the specimens were stored before extraction.

Effective Specific Gravity

Several mixtures having different asphalt contents were prepared for each aggregate blend and tested by the procedure: *Maximum Specific Gravity of Bituminous Mixtures by Vacuum Saturation Procedure*, by J. M. Rice, ASTM Special Technical Publication No. 191, June 1953, pp. 43–61, or by test method: ASTM D 2041–64T. The maximum specific gravities were used to compute the effective specific gravities of the aggregates listed in table 2. The following formula was used:

$$\text{Effective specific gravity} = \frac{100}{\frac{100+P}{M} - \frac{P}{g}}$$

Where:

P = Percent asphalt, aggregate basis.
M = Maximum specific gravity of the mixture.
g = Specific gravity of the asphalt.

Surface Area of the Aggregate

Surface areas, corresponding to those determined by California, were computed from the gradations listed in table 2. The formula used was, as follows:

$$\text{Surface area (ft.}^2\text{ per lb. of aggregate)} = 2+0.02a+0.04b+0.08c+0.14d+0.30e+0.60f+1.60g$$

Where a, b, c, d, e, f, and g are percentages of total aggregate passing sieve sizes Nos. 4, 8, 16, 30, 50, 100, and 200, respectively.

Bitumen Index

California defines bitumen index as pounds of asphalt binder per square foot of surface area. The formula used for computing the bitumen index as given in tables 3, 8, 9, and 10 was:

$$\text{Bitumen index} = \frac{\text{Percent asphalt, aggregate basis}}{100 \times \text{surface area}}$$

Rate of Airflow

Figure 3 shows the apparatus used in determining air permeability. The primary components are:

- A mold assembly for containing the test specimen.
- A sensitive needle valve for controlling the rate of airflow to provide a constant pressure differential. Airflow is produced by use of a laboratory vacuum or pressure line.
- A sensitive manometer for measuring pressure differentials of up to 2 inches of water.
- Three calibrated flowmeters of different capacities for measuring rates of airflow ranging from 2 to 9,100 milliliters per minute. Valves are provided for cutting off the two flowmeters not in use.

Use of the apparatus when a vacuum line is employed for producing airflow is illustrated in figure 4. The test mold consists of 3 parts:

- A metal cylinder 4 inches in diameter, open on both ends, that serves as a support for the test specimen.
- A metal cylinder 4 inches in diameter, open at the bottom, that rests on top of the test specimen; this cylinder is identified as the test cap.
- A metal cylindrical jacket slightly larger than 4 inches in diameter and containing a 4-inch diameter rubber triaxial sleeve as an inner liner. The support, specimen, and cap are confined by this inner liner. Connections are provided on the metal jacket for applying pressure or vacuum by a hand bulb to inflate or deflate the rubber liner. A pressure gage is also included.

The vertical face of the specimen normally is sealed with paraffin or some other suitable material prior to testing for permeability. To facilitate the placement of the sealed specimen into the test mold, the rubber membrane is deflated. After placing the support, specimen, and cap inside the jacket, a weight of 2,000 grams is placed on top of the cap, and air pressure of 2 p.s.i. is then applied to the rubber membrane to force it against the specimen and, finally, the hose connections shown in figure 4 are completed.

The procedure for measuring rate of airflow for a particular pressure differential is to open the valve for the most sensitive flowmeter and, keeping the other two closed, regulate the rate of airflow with the needle valve to maintain the desired pressure differential. When the desired pressure differential is obtained, the level of the floating ball is read and recorded and the rate of airflow can be determined from the calibration chart. If the level of the ball exceeds the maximum reading of the flowmeter, a flowmeter of higher capacity should be used.

Permeability

Most of the rates of airflow were determined for pressure differentials of ½, 1, and 2 inches of water. Permeability, in fundamental units, was computed from the conventional formula given by Hein and Schmidt (*9*):

$$K = \frac{\mu \bar{Q} L}{A(p_1 - p_2)}$$

Where:

K = Permeability per cm.^{2}, cm.^{3} per sec.
μ = Viscosity of air in poises.
$\bar{Q}$ = Rate of flow, cm.^{3} per sec.
L = Height of sample, cm.
A = Area of sample, cm.^{2}
$p_1 - p_2$ = Pressure differential, dynes per cm.^{2}

All specimens used in this study were 4 inches in diameter and 2½ inches high. Laboratory temperatures during testing were not recorded but ranged from about 80° F. to 90° F. Based on data from the *Handbook of Chemistry and Physics*, 43d ed., 1961–1962, the average viscosity of air was estimated to be 1.853×10^{-4} poises. Therefore, the formula reduces to:

$$K = \frac{0.974 \times 10^{-10} \times F}{P}$$

Where:

F = Rate of airflow, ml. per min.
P = Pressure differential, inches of water.

This simplified formula was used in computing the permeabilities shown in tables 4, 5, and 6.

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NEW PUBLICATIONS

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds

The third issue of *Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds* (May 1965), is now for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for 75 cents. This publication is a compendium of information on Highway Research and Development studies financed with 1½-percent Planning and Research funds approved by the Office of Research and Development, Bureau of Public Roads. Information is included for each State for a 12-month period on projects approved for the calendar year 1965, or for the fiscal year 1965, which began July 1, 1964.

The information has been grouped according to seven broad study areas, and data are also presented on the objective of each study, the conducting agency, and the funding for each study. The seven areas are: Determination of the Future Role and Type of Facilities Needed for Highway Transportation; Reduction of Highway Accidents; Increase the Capacity of Urban Roads and Streets; Develop Method for Designing and Evaluating Highway Pavements and Structures; Reduction of Costs of Drainage Installations; Reduction of Construction, Maintenance, and Administration Costs; and Improve Capability for Conducting Research and Effect Rapid Utilization of Research Results.

This publication also contains a listing of the projects for the fiscal year 1965 in the American Association of State Highway Officials National Cooperative Highway Research Program (NCHRP). Sponsored by AASHO and administered by the Highway Research Board of the National Academy of Sciences, these projects are financed from a pool of funds consisting of 5 percent of the 1½-percent Federal-aid funds of the participating State highway departments.

Urban Planning Publications

Two new publications prepared by the Urban Planning Division, Office of Planning, Bureau of Public Roads, are available from the Superintendent of Documents, Washington, D.C., 20402, at the price listed for each: *The Role of Economic Studies in Urban Transportation Planning*, by Joseph P. Meck, 45 cents; and *Traffic Assignment and Distribution for Small Urban Areas*, by the Urban Planning Division, $1.00.

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A general guide to the form for economic studies for transportation planning is presented in *The Role of Economic Studies in Urban Transportation Planning.* This publication is not a manual of procedures but rather contains a general introduction to methods and procedures for forecasting economic activity. It is expected to be useful to those having little or no training in economics but who are concerned with transportation planning. Specifically the publication is expected to:

- Provide assistance and guidance to those responsible for reviewing economic forecasts required as part of the comprehensive transportation planning process in urbanized areas.
- Identify some of the important dynamic factors that influence the process of economic development of urbanized areas so that a better understanding of that process can be obtained.
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- Provide a listing of useful sources of data for economic studies.

Traffic Assignment and Distribution for Small Urban Areas

The manual, *Traffic Assignment and Distribution for Small Urban Areas*, explains and illustrates the theory and use of a system of analytical procedures and computer programs for assigning and distributing trips to transportation systems with the IBM 1620 (60K) electronic computer. The procedures described are designed to process a basic set of trip cards. They allow the computation of surveyed trip length frequency, distribution of trips between zones by the gravity model formula or Fratar method, and assignment of trip interchanges to an existing transportation network.

The procedures also accommodate estimates of future trip production and trip attraction and provide for assigning the trip distribution to a proposed transportation network. As the analytical procedures described in the volume are mechanized by a highly flexible series of computer programs, which utilize the IBM 1620 (60K) electronic computer, they should be particularly useful to persons processing and analyzing urban transportation data.

This battery of computer programs was developed by several State highway departments and the Bureau of Public Roads, and is generally applicable to urban areas having populations up to 150,000 persons. To utilize the computer programs for this system, the study area must be described by not more than 699 nodes, of which 200 may be zone centroids. The maximum traveltime that can be accommodated is 99 minutes.

U.S. GOVERNMENT PRINTING OFFICE: 1965

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Interstate 95–6, north of Old Belgrade Road, near Waterville, Maine.

View is toward the south on southbound roadway and shows the use of split profile on side hill location.

IN THIS ISSUE: *Article on why drivers choose an expressway or primary route*

IN THIS ISSUE

Attitudes of Drivers Determine Choice Between Alternate Highways, *by R. M. Michaels* 225

Illumination Variables in Visual Tasks of Drivers, *by H. R. Blackwell, R. N. Schwab, and B. S. Pritchard* 237

New Publications 236

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Contents of this publication may be reprinted. Mention of source is requested.

Attitudes of Drivers Determine Choice Between Alternate Highways

Reported [1, 2] by RICHARD M. MICHAELS,
Science Advisor, Program Management Staff

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

The research information presented in this article is based on a study of the factors that influence a driver's choice of alternate routes. Through the study in which the attitudes of drivers toward two highways were measured, an attempt was made to determine the utility of attitude scaling methods for predicting the choice. Establishment of such a subjective measure was sought for use in highway design, traffic planning in general, and predicting the use that will be made of new and improved highways. The author believes that the data collected show that this subjective method of evaluating route choice is a simple and effective means of predicting use of highway facilities.

In addition to the attitudes of the drivers, traffic characteristics of the routes were measured and the tension generated on each was determined. Nine test drivers were used for the tension tests. The routes employed were 47-mile sections of an expressway design toll road and a parallel rural primary highway. Drivers were sampled entering and exiting on both highways. A summated rating attitude scale was administered to a sample of 3,259 drivers. Descriptive information was obtained about the driver, his trip, and travel habits. Analysis of results showed that these drivers held stable attitudes that clearly differentiated between the routes. Direct measurement of driver attitudes seems to be a far better predictor of route choice than any descriptive information about the drivers or their driving habits.

In addition, the results provide a means of rationalizing the attraction of traffic to an expressway on the basis of drivers who seek to minimize tension in driving. The data suggest that total stress incurred in driving is a more important determinant of route choice than either operating costs or traveltime costs. A model of route choice and attraction of traffic is proposed based upon tension generation that can be related to traveltime data. Analysis of this research shows that drivers evaluate the use of alternate highways in a rational, though subjective, fashion. Such evaluation, however, seems to be very independent of the usual monetary plans often used to measure highway benefits and costs.

Introduction

WHENEVER a driver is provided alternate routes, he must make an evaluation of the benefits and costs of using each in order to make a choice. If he knew nothing about available alternate highways or did not make an evaluation of them, his choice would be random. Because drivers do not operate in a random manner, it seems reasonable to assume that they learn the characteristics of the highways and out of this learning develop a basis for evaluation of alternate routes. A driver's choice thus becomes dependent on the diverse characteristics of the alternates relative to his trip objectives, and these determine stable choice behavior. This behavior is of considerable significance both in determining the use of highway facilities and the benefits a driver derives from them.

Three major factors have been developed to account for the patterns of choice that a driver makes between alternate highways. The first is the time savings obtained by taking one route instead of the other. The second is the direct and indirect operating cost savings obtained by taking one route instead of the other. The third is the comfort and convenience savings obtained by taking one route instead of the other.

In general, traveltime savings have been the dominant criterion of use of alternate facilities; the best predictor being the traveltime ratio. In both rural (*1, 2*)[3] and urban studies (*3, 4*) a driver seems to choose routes that provide significant time savings, even though he may have to drive a longer distance. Discussions in all these studies imply that the driver values time directly and, hence, scales that variable. From an economic standpoint, a considerable effort has been made to determine the dollar equivalent of this time scale. For passenger car drivers these attempts have not been particularly successful (*5*). The relation of operating cost to choice by a passenger car driver seems to be weak (*6*). Either the driver does not evaluate operating cost differences or these differences are insignificant. When related to the total costs of a trip, operating cost differences between alternate routes may be very trivial for the passenger car driver.

In addition to these physical measurements, the purely subjective concept of comfort and convenience has been developed. This has generally been described qualitatively as the ease of driving or freedom of movement. Claffey (*6*) has scaled this factor in terms of the changes in speed imposed on the driver and, hence, counted the impedances to movement. Michaels (*7*) has differentiated among highways on the basis of the tension aroused in a driver from traffic and geometric design features. His results indicate that tension reduction is the greatest single saving accruing to a driver who chooses an expressway over a parallel uncontrolled-access highway, and the driver seems to subjectively evaluate alternates in conformity to the tension induced on each.

Although the research reports on the problem of use of alternates have described what traffic does, little research has been carried out on driver perception of alternate routes available (*3*). Further, no attempts have been made to measure on a quantitative scale the evaluations a driver makes or his relation of these evaluations to choice of routes. Thus, no reliable way now exists to predict usage of facilities except by empirical studies of traffic.

Regarding any benefit in analysis of highway facilities, obviously, drivers evaluate on a predominantly subjective basis. No economic determination seems feasible unless the

[1] Presented at the 44th annual meeting of the Highway Research Board, Washington, D.C., January 1965.

[2] Prepared in cooperation with the Maine State Highway Commission and the Maine Turnpike Authority. Ralph H. Sawyer, formerly Planning and Traffic Engineer of the Maine State Highway Commission, and William B. Getchell, Jr., formerly Executive Director of the Maine Turnpike Authority, both now dead, contributed invaluable assistance and counsel for the research on which this article is based. The author also was assisted in obtaining data by Daniel Bridges and Harold C. Wood, Jr., both employees of the Bureau of Public Roads.

[3] References indicated by italic numbers in parentheses are listed on page 236.

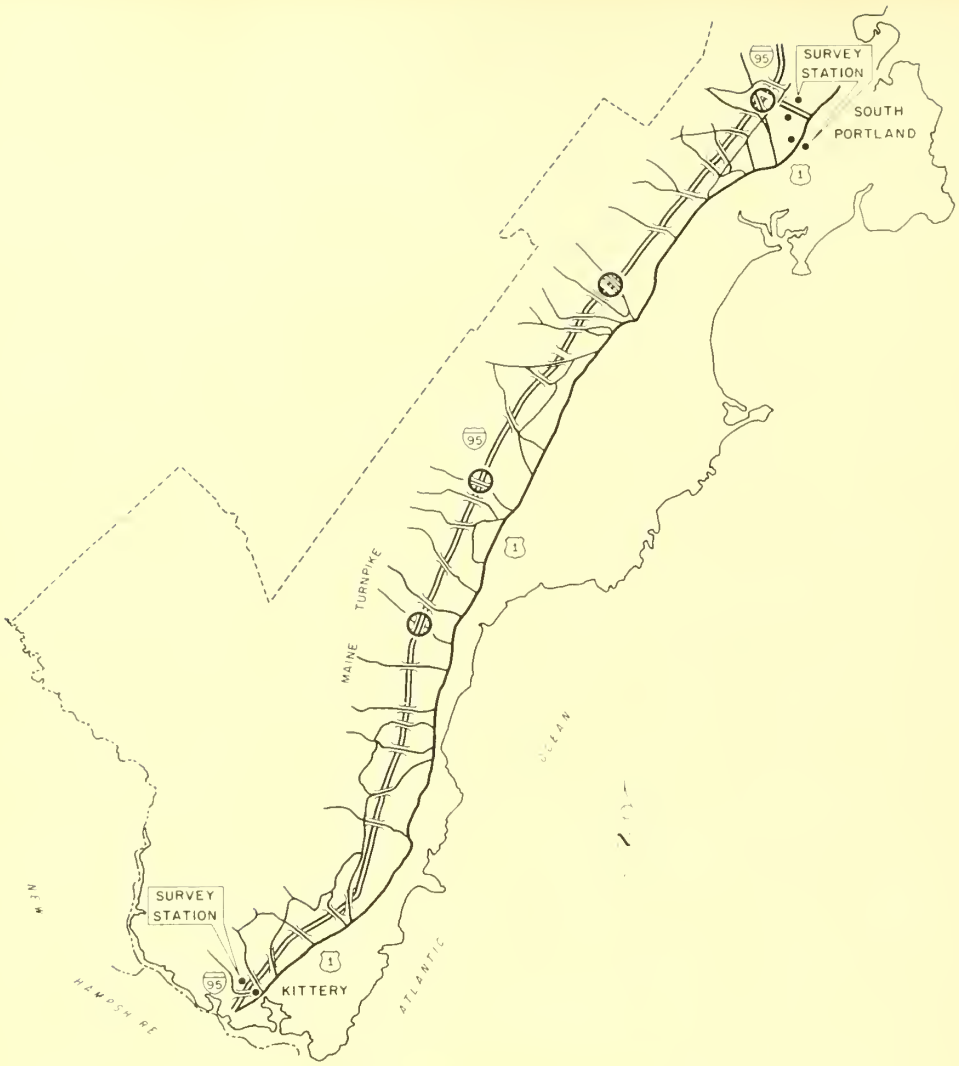

Figure 1.—Map locating study routes.

scale of value drivers use and its relation, if any, to dollars is known.

Considering the problem of selection of alternate routes, a reasonable assumption is that choice will be based upon what the driver has learned about the alternate. Either directly or indirectly, a driver must develop some stable evaluations. That is, he must have some predisposing views toward the routes or his choices would be random. These predisposing views are, by definition, the attitudes an individual holds toward some object or process. If route choice is rational, then a direct measure of a driver's evaluation should be his attitudes toward the alternate. By determining the intensity of these attitudes toward a pair of highways, it should be possible to determine how these attitudes are related to the characteristics of the highways and the choices drivers make.

To achieve these objectives, however, it is first necessary to determine whether a stable set of attitudes exists toward highways of different characteristics. Second, it is necessary to determine whether these attitudes depend on the characteristics of the drivers, which are relatively permanent, or upon the characteristics of a particular trip that would cause highly variable attitudes. In this context, the study discussed here was developed. The aim was to test the hypothesis that drivers on each of two highways had significantly different attitudes toward the two highways and that these attitudes were based on the more enduring characteristics of the routes and the drivers.

Development of the Attitude Scale

The attitude scaling technique employed in this study was the Method of Summated Ratings. It employs a series of direct statements to which the respondent expresses the extent of his agreement. An example of such a statement might be, "A road with many hills and curves is interesting to drive." The test subject then responds in one of five categories ranging from "strongly agree" to "strongly disagree." A score of 0, 1, 2, 3, or 4 is given to his response, according to the category chosen, a score of 2 being neutral. Thus, by using a set of such items, a total attitude score can be obtained for any test subject toward the road under study.

The general procedure for preparing such an attitude battery is described by Edwards (*8*). In the study reported here, it was decided to compare attitudes on a toll road and a rural primary road as these are two of the more common that a driver has to choose between, and yet they have radically different design characteristics. To develop the final items for the attitude scale, 61 statements were initially prepared. They described a variety of characteristics of a rural primary road and an expressway, both positive and negative. They were presented to 260 staff members of the Bureau of Public Roads. Instructions given were:

"Place yourself in a hypothetical situation of having the choice of two routes for home to work trips: (1) a controlled-access toll road, and (2) a parallel free-access primary roadway. The toll on the turnpike is $1. The trip is 30 miles on both routes. Assume that the primary route is similar to U.S. 1 between Baltimore and Washington, or between Alexandria and Woodbridge.

"The attached questionnaire is designed to elicit attitudes toward these two types of highways. You should respond to each statement in terms of your own personal feelings, checking one of the five categories that range from strongly agree to strongly disagree."

Some basic objective information was obtained about the respondents, including age, sex, and the percentage of time they would choose the toll road. Adding the last item permitted an initial check on the validity of the final scale, for it was hypothesized that those responding most positively to expressway items would be those most likely to use that facility. All items were scored in terms of favorability toward the expressway. The returns were then analyzed according to the standard procedure in which the highest scoring quarter of the sample was compared with the lowest scoring quarter; well over half the items significantly differentiated between the two highways. The final battery was composed of 18 items, from the original group of 61, that were the most discriminating between the groups having high and low scores.

A further analysis was made on this pretest group. The attitude scores were correlated with the respondents' percentage of choice of the toll road. The two distributions were dichotomized and a phi coefficient was computed. The correlation coefficient was +0.52 between attitude scores and choice of routes. Thus, it was reasonable to conclude that, in this hypothetical situation, a stable set of attitudes existed toward the two types of highways that was significantly related to the choice of routes the respondents would make.

In addition to the final attitude battery, a questionnaire was included to obtain some basic descriptive information about the respondent's trips so that the attributes of the driver and his trips could be related to his attitudes. These items were to provide a means for testing the stability of the attitudes and fell into three basic categories. The first was the characteristics of the driver and his vehicle, including age and sex of driver, and age of car. The second was the characteristics of the trip, including purpose, number of car occupants, the driving time already completed, and driving time to be completed.

The third was the descriptive information of the driver's estimate of the frequency with which he made this kind of trip and the frequency with which he used the alternate route.

The item on driving time was included because no statements relating to traveltime alone were in the attitude battery. In the sample used to develop the scale, time was not a discriminating factor between the groups scoring high and low. By treating traveltime as an independent variable, subjective estimates of driving time could be related to the respondent's attitudes toward the routes. Obviously, if traveltime were a dominant criterion of choice, then a correlation should exist between the driver's attitude toward the route and the duration of the trip that he was undertaking. By using this approach, an independent test could be made of a driver's choice of routes and of traveltime.

Selection of Test Location

In considering a pair of roads of sharply different characteristics between which a driver might choose, the ideal would be a pair that had a common beginning and a common terminus. In addition, the pair should be long enough to permit a meaningful choice by the driver. A pair of highways that fit these requirements is the Maine Turnpike between Kittery and South Portland and the parallel rural primary, U.S. 1, which has been studied extensively over the past decade (*1*, *2*). The sections are approximately the same length, about 45 miles. At the Kittery end, the choice of route is a simple one for the driver, for the connection is a Y. At the South Portland end, U.S. 1 and the Turnpike join again. A map of the two roads is shown in figure 1.

The characteristics of both routes are typical of a modern toll road and a rural primary. The Turnpike is a 4-lane divided highway on which interchanges are spaced 5- to 15-miles apart; they generally have been built to Interstate design standards. U.S. 1 varies from 2- to 4-lanes and passes through several small towns and undeveloped countryside. Access is not controlled, and the route has a variety of traffic control devices.

Procedure

A survey team of nine men was used. The sampling schedule was set for daylight hours between 8 a.m. and 5 p.m., and was effected at both ends of each highway. During the first 4 hours, vehicles were stopped as they entered the test sections; during the next 4 hours, they were stopped as they left the test sections. Samplings were obtained from north and south ends of both routes, but drivers were not stopped twice on the same trip. By counterbalancing the order, an approximately equal sampling of drivers entering and exiting at both ends of the two highways was obtained.

To obtain the most stable attitudes toward the routes under study, only Maine or New Hampshire drivers were stopped. No fixed procedure was established for stopping a particular vehicle. The complexities of traffic and the fact that only two interviewers were at each station precluded any formal sampling procedure. However, by extending the sampling period for more than 30 days, it is believed that most biases were eliminated.

When a driver was stopped, a common set of instructions was given:

"Good morning. We are doing research on why drivers pick particular roads for their trips and would like to enlist your assistance. We have a questionnaire that we would like you to complete, which will take about 5 minutes of your time. If you can spare that time, we would appreciate it."

If the driver agreed, the attitude form was handed to him and the instructions for filling it out were read with him. When the interviewer and the driver were satisfied as to what was wanted, the interviewer withdrew and the driver completed the attitude questionnaire. When finished, he handed the form back to the interviewer who then asked the objective questions and marked the verbal replies on a coding sheet. The two parts of the form had a common number so that both parts of the survey could be combined subsequently.

Speed and volume measurements

In addition to the attitude survey, traffic measures were taken on the two routes. Rather complete volume counts were made daily for both the Turnpike and U.S. 1. On U.S. 1, volume counters were placed at three locations for hourly traffic counts. On the Turnpike, volume was sampled at four locations during several different time periods. In addition, a radar speed meter recorded daily samples of traffic speed on both routes. Thus, a fairly complete record of the traffic characteristics on both test sections was obtained during the period of the study.

Tension measurements

The galvanic skin reflex (GSR) test was employed to obtain tension measurements on both the Turnpike and U.S. 1. During the 1-month study each of the interviewers was used as a test subject and drove both routes twice in both directions. The procedure outlined in previous reports (*7*, *9*) was employed.

Results

During the 4 weeks of surveying on both routes, a total sampling of 3,259 different drivers was obtained. No significant differences were noted between drivers sampled at the two ends of the test routes. Also, no differences were noted between drivers sampled on entering the test sections and those leaving them. Hence these data were pooled. As shown in table 1, approximately the same

Table 1.—Attitudes of drivers toward the Maine Turnpike and U.S. 1

Sex of drivers	Maine Turnpike			U.S. 1		
	Number sampled	*Mean attitude score*	*Standard deviation*	*Number sampled*	*Mean attitude score*	*Standard deviation*
Male	1,138	41.33	9.40	1,039	32.09	9.56
Female	482	38.52	9.54	600	30.26	8.65
Total	1,620	----------	----------	1,639	----------	----------

Table 2.—Distribution of drivers sampled on Maine Turnpike and U.S. 1, by sex

Sex of drivers	Maine Turnpike		U.S. 1		Total	
	Number sampled	*Percent*	*Number sampled*	*Percent*	*Number sampled*	*Percent*
Male	1,138	70.4	1,039	63.4	2,177	66.7
Female	482	29.6	600	36.6	1,082	33.3
Total	1,620	100.0	1,639	100.0	3,259	100.0

Table 3.—Age of vehicles on the Maine Turnpike and U.S. 1, by sex of driver

Vehicles		Vehicle distribution by drivers sampled on—			
Age	Sample	Maine Turnpike		U.S. 1	
		Male	Female	Male	Female
Year	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Less than 1	18.6	22.5	22.1	16.7	13.2
1–3	39.3	45.9	39.6	34.2	35.9
4–6	26.2	20.1	27.1	28.1	32.8
More than 6	15.7	11.4	11.1	21.3	18.0

Table 4.—Analyses of variance of attitudes of male and female drivers toward Maine Turnpike, based on age of drivers and vehicles

Source of variance	Sum of squares	Degree of freedom	Mean squares	*F*, ratio	Probability (*F*)
Male Drivers					
Driver age	656.53	3	215.51	2.468	<0.05
Vehicle age	996.48	2	498.24	5.706	<0.01
Age × vehicle	243.35	6	40.56		(1)
Residual	99,454.14	1,139	87.32		
Total	101,341.50	1,150			
Female Drivers					
Driver age	464.30	3	154.80	1.543	(1)
Vehicle age	263.10	2	131.55	1.312	(1)
Age × vehicle	418.42	6	69.74		(1)
Residual	48,342.20	482	100.30		
Total	49,488.02	493			

[1] Not significant.

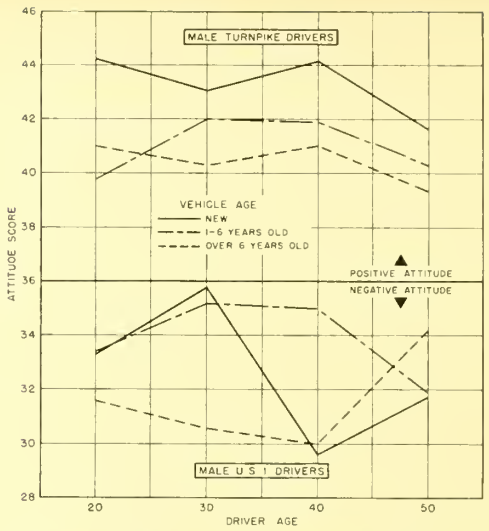

Figure 2.—Male driver attitudes toward turnpike as function of driver and vehicle age and travel route.

Table 5.—Analyses of variance of attitudes of male and female drivers toward U.S. 1, based on age of drivers and vehicles

Source of variance	Sum of squares	Degree of freedom	Mean squares	*F*, ratio	Probability (*F*)
Male Drivers					
Driver age	2,532	3	844.0	9.58	0.01
Vehicle age	629	2	313.5	3.56	0.05
Age × vehicle	1,390	6	231.7	2.62	0.05
Residual	86,299	980	88.1		
Total	90,850	991			
Female Drivers					
Driver age	1,148	3	382.7	5.50	0.01
Vehicle age	755	2	377.5	5.42	0.01
Age × vehicle	722	6	120.3	1.73	(1)
Residual	42,605	604	69.5		
Total	45,230	615			

[1] Not significant.

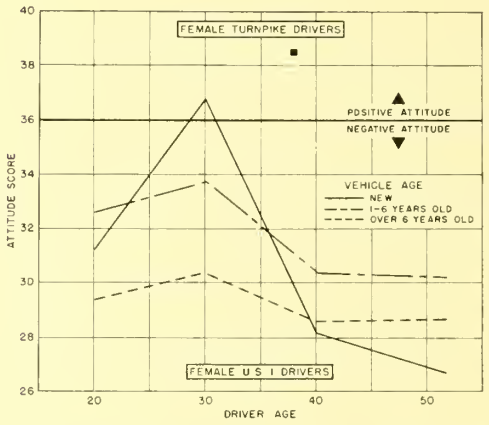

Figure 3.—Female driver attitudes toward turnpike as function of driver and vehicle age and travel route.

number of observations were taken on both routes. This, of course, does not represent the distribution of traffic but only the method of sampling on the two highways.

Fourteen percent of the drivers stopped declined to participate in the survey. This percentage was the same on both routes. In addition, approximately 6 percent of the drivers stopped had been interviewed before. As might have been expected, the percentage of repeats from the first week to the last week rose on U.S. 1 from 1.9 percent at the end of the first week to 5.7 percent the third week. On the Turnpike, the figures rose from 0.8 percent, at the end of the first week, to 10.3 percent the third week.

Attitude Survey

The Turnpike was used as a reference for assigning a quantitative score to the responses when the attitude questionnaires were scored. Thus, all statements about U.S. 1 that reflected a positive attitude toward it were given a 0 score for the category of "strongly agree" and a score of 4 for the response of "strongly disagree." For those items that were unfavorable statements about U.S. 1, strong agreement was scored as 4 and strong disagreement as 0. Statements about the Turnpike were scored in the obvious reverse manner. Thus, the total score of a respondent was interpreted to reflect his attitude toward the Turnpike. The scores on each of the items and the descriptive information obtained from the interview were placed on punchcards, and all of the basic analyses of the attitude sampling was performed by a computer.

A summary of the attitudes of drivers on each route is shown, by sex, in table 1. The higher the score, the more positive the feelings of the drivers toward the Turnpike. A score of 36 indicated a neutral attitude toward the Turnpike. As shown in table 1, significant differences were stated for choosing between the two highways. Drivers on U.S. 1 had negative attitudes toward the Turnpike, and Turnpike users had positive attitudes toward it. Also, the differences stated by the sexes were significant. The male drivers on the turnpike were significantly more positive toward the Turnpike than the female driver. On U.S. 1, the male driver, although having a negative attitude toward the Turnpike, was less negative than the female driver. The attitudes of male and female drivers on both routes were significantly different from neutral. Thus, it is reasonable to conclude that use of the attitude scale showed a differentiation between the users of the two highways.

The sex distribution of the drivers on the two routes was analyzed and, as shown in table 2, two-thirds of the total sampling of drivers was male. More significant, however, is the difference between the proportion of male or female drivers on the two routes. Significantly more female drivers traveled U.S. 1 than the Turnpike. Comparison of this sex distribution with attitudes toward the Turnpike (table 1) indicates a significantly less positive attitude of the females than the males toward the Turnpike. Therefore, it was concluded that a correlation existed between the attitudes held by the two sexes toward the highways and the actual choices they made.

The third category under the driver and vehicle characteristics concerns that of vehicle age. The percentages of vehicles on each

route, by their age, and by sex of their drivers is shown in table 3. Two inferences may be made from this table: First, in this sample, vehicles driven by females were older than those driven by males. Second, and more significant, the percentage of older vehicles on the Turnpike was considerably less than those on U.S. 1.

Drivers in the sampling on both routes were compared for age differences. In relation to attitudes toward the two highways, rather clearcut differences existed. An analysis of variance was performed for both driver age and vehicle age, the attitude scores being the dependent variable. The summary tables for males and females using the Turnpike are shown in table 4 and for those on U.S. 1, in table 5. Both driver age and vehicle age were statistically significant in every analysis except for the female drivers on the Turnpike. In figures 1 and 2 the mean attitude scores as a function of age are shown for all conditions. Vehicle age is the parameter in these curves. As shown for the male drivers, attitudes toward the Turnpike became less positive as their age increased. Vehicle age also had a clear effect on the attitudes. Thus, the newer the automobile, the more positive was the attitude toward the Turnpike. In general, the same results were obtained for the female drivers on U.S. 1; that is, there was a definite ordering of attitudes by age of vehicle and driver. A peak in attitudes toward the Turnpike seemed to occur in the age range of 25 to 35, after which drivers' attitudes became more negative toward the Turnpike. No significant differences were noted for the female driver on the Turnpike. From these analyses it was concluded that attitudes toward the alternate highways were significantly dependent on the stable characteristics of the driver and his vehicle. Analyses of these results further indicate that attitudes toward alternate routes were very stable, evolving partially out of the enduring characteristics of the driver and his vehicle.

Attitudes and Trip Characteristics

The second class of relations to a driver's attitude concerned the characteristics of the specific trip during which the driver was sampled. The objective of this analysis was to determine whether the attitudes toward the two highways as markedly modified by the purpose of the trip, the number of occupants in the vehicle, and the traveltime associated with the trip. Analysis showed that no significant relations existed between either the trip purpose or the number of occupants in the vehicle and the driver's attitude toward the Turnpike. Similarly, the relation between subjective estimates of trip duration was unrelated to driver's attitude toward the Turnpike. Thus, the results of this analysis on the characteristics of the specific trip indicate that a driver's attitude was independent of the specific trip. The choice, then, between alternates was made on a basis of stable and preexisting attitudes toward the different types of highways.

The results relevant to traveltime should not be interpreted to mean that there were no differences in the distribution of trip durations on the two highways. Table 6 contains the frequency distributions for the sample. These time vaules are subjective estimates of the time already spent driving as well as being estimates of the time required to complete the trip. Therefore, the longer the trip, the more likely it was to be made on the Turnpike. Thus, approximately 32 percent of all drivers sampled on the Turnpike had been traveling for less than one-half hour and 54 percent had more than 1 hour left to drive. But on U.S. 1, 70 percent of the drivers had been driving for less than one-half hour and only 25 percent needed more than another one-half hour of driving to complete their trip. A slightly different presentation in figure 4 shows the percentage distribution of remaining triptime for drivers who had just started their trips. Only 15 percent of those on U.S. 1 expected to be driving for more than one-half hour, whereas 71 percent of the drivers starting their trips on the Turnpike expected to drive for more than one-half hour. Thus, the drivers on longer trips were the ones that tended to gravitate toward the Turnpike

A clearer understanding of the effects of triptime and attitudes can be obtained by examining reports of only those travelers on both routes who had approximately common origins and destinations. If only those Turnpike drivers are selected who had been traveling for less than one-half hour and who had between one-fourth hour and 1 hour left to travel, they could be compared with U.S. 1 drivers who also had been traveling for less than one-half hour but who had between one-half hour and 2 hours more to drive. Obviously, drivers who chose U.S. 1 sacrificed time. The attitudes of male drivers of different ages who chose the Turnpike were compared; the scores are shown in table 7. There were no significant differences among the ages of Turnpike drivers; whereas on U.S. 1, choice of the Turnpike decreased significantly when the drivers were older. However, the U.S. 1 driver always had a significantly negative attitude toward the Turnpike. Thus, it is concluded that for trips having common origin and destination, the driver's choice between the two routes was related mostly to his attitude toward the alternate. For drivers on U.S. 1, this showed that they chose the rural primary route instead of the expressway although this choice increased traveltime 30 percent.

The sample was also analyzed in relation to the frequency with which drivers made trips between South Portland and Kittery. Trip frequency was defined in three categories: Less than 1 trip a year, 1 to 12 trips a year, or more than 1 trip a month. The distribution was computed for both the Turnpike and U.S. 1 and for the two sexes. The percentage of the total sampling on each route for the two sexes and the trip frequencies are shown in table 8. In the Turnpike sampling, the majority of the drivers made the trip more than once a month. On U.S. 1, however, the majority of the drivers made the trip between once a year and once a month. A chi-square test was used to test the differences between the number of trips made on the Turnpike and those on U.S. 1, and the differences between the distributions were significant. When trip frequency increased to more than one trip a month, the proportion of these trips made on U.S. 1 decreased and the proportion on the Turnpike increased. This may indicate that the Turnpike exerted an attraction for drivers as the frequency with which they traveled between Kittery and South Portland increased.

The attitudes of drivers toward the two routes were also analyzed as a function of frequency with which trips were made between South Portland and Kittery. The mean attitude scores are shown in table 9. Because of the significant differences among ages of drivers, the data also are separated by that variable. Two inferences may be made: First, the influence of age is the same as discussed previously. Second, as a function of trip frequency, a consistent and significant increase occurred in the average attitude score of both male and female drivers toward the Turnpike. In addition, the drivers on U.S. 1, although having negative attitudes toward the Turnpike, tended to have a change in attitude,

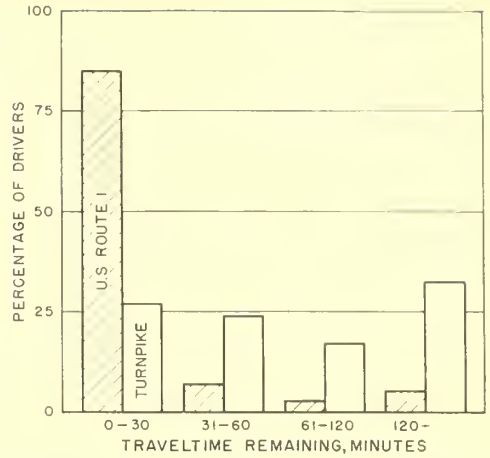

Figure 4.—Remaining trip time after driving less than 30 minutes, percentage distribution.

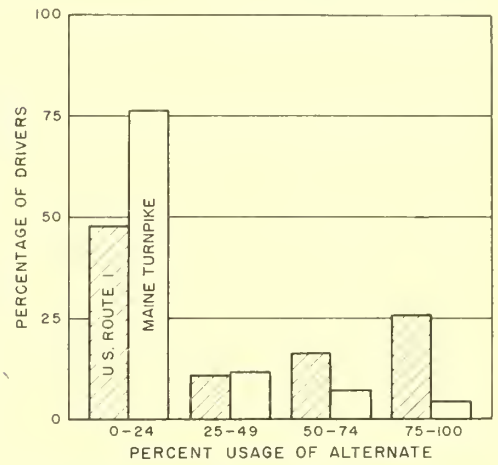

Figure 5.—Frequency of usage of alternate routes by drivers sampled, both routes.

Table 6.—Distribution of driving times for drivers traveling on Maine Turnpike and U.S. 1

Driving time completed, minutes ↓	Maine Turnpike						U.S. 1					
	Driving time left, minutes						Driving time left, minutes					
	Less than 15	15-30	31-60	61-120	More than 120	Total	Less than 15	15-30	31-60	61-120	More than 120	Total
Less than 15:												
Male	55	12	45	35	68	215	190	146	29	16	29	410
Female	4	2	22	11	23	62	136	96	17	4	7	260
15-30:												
Male	16	40	32	32	66	186	91	146	31	14	19	301
Female	8	23	15	24	25	95	94	92	20	13	6	195
31-60:												
Male	35	43	27	34	42	181	41	51	12	11	14	129
Female	13	21	12	8	13	67	12	34	9	5	6	66
61-120:												
Male	31	44	33	59	85	252	25	27	17	16	20	105
Female	10	28	20	35	35	128	7	20	7	8	3	53
More than 120:												
Male	36	64	38	72	173	383	14	16	10	26	48	114
Female	12	23	22	32	64	153	8	16	4	1	16	45
Cumulated total:												
Male	173	203	175	232	434	1,217	361	386	99	83	130	1,059
Female	47	97	91	110	160	505	227	266	57	31	38	619

Table 7.—Mean attitude scores for male drivers whose trips had approximately common origins and destinations

Driver age	Mean attitude scores for male drivers on—	
	Maine Turnpike	U.S. 1
Less than 24	-------	35.37
24-34	42.47	34.22
35-44	43.32	33.72
More than 44	41.73	29.90

approaching neutrality, toward the Turnpike as trip frequency increased. Thus, as trip frequency increased, a general shift to more positive attitudes toward the Turnpike occurred. This result offers further evidence that a driver's attitude toward the two highways shifted, on the basis of his driving experiences on both of the routes, toward favoring the expressway.

A final general analysis was made concerning the extent of utilization of the alternate routes by drivers. Each driver sampled was asked what percentage of time he used the other route for his trips. The percentage of the drivers sampled, who used the alternate route a specific percentage of the time, is shown in figure 5. Because of no differences in data for male and female drivers, all the data were combined. The drivers sampled on the Turnpike rarely used U.S. 1—only 12 percent of the sampling of Turnpike drivers used U.S. 1 for more than half their trips. But drivers sampled on U.S. 1 frequently used the Turnpike—42 percent used it for more than 50 percent of their trips. This usage also indicates an attraction of drivers toward the Turnpike.

Attitude Scale

The attitude scale employed in this study was composed of two classes of items. One classification of the statements was by their reference to either the Turnpike or U.S. 1, and the other was according to whether they were favorable or unfavorable. Hence, the the items in the attitude scale can be classified in a 2 by 2 matrix. In addition, the total attitude score was arbitrarily scored in relation to the Turnpike—a negative statement about U.S. 1 was interpreted as being favorable toward the Turnpike; conversely, a positive statement toward U.S. 1 was interpreted as being negative toward the Turnpike. An item analysis of the attitude scale was made to determine the effects of these different kinds of statements. A sampling of data on the respondents was selected at random on the basis of the percentage of the time they used the alternate route. Each item was classified as to whether it referred to the Turnpike or U.S. 1 and as to whether it was a favorable or unfavorable statement. In these classes, the score value was determined by the extent of agreement with the item itself by the respondent. Thus, a score value of more than 2 indicates agreement with the item, regardless of whether it is favorable or unfavorable. Conversely, a score value of less than 2 indicates disagreement with the statement. In tables 11 and 12, the data are shown for the male drivers.

As shown in table 10, regardless of the route upon which they were sampled, and regardless of the percentage of their trips on the Turnpike, drivers responded positively to favorable statements about the Turnpike. In response to unfavorable statements, drivers sampled on the Turnpike, regardless of their frequency of use, disagreed with the statements and, hence, provided a positive response toward the Turnpike. Drivers on U.S. 1, however, strongly agreed with the negative Turnpike statements if they were infrequent users of the Turnpike and strongly disagreed if they were frequent users. Thus, there was a significant shift in response to the negative statements by U.S. 1 drivers as a function of the frequency with which they used the Turnpike.

Conversely, as shown in table 11, drivers sampled on the Turnpike were essentially neutral in their responses to favorable statements about U.S. 1, regardless of whether they were frequent or infrequent users of the Turnpike. Drivers sampled on U.S. 1, responded to the favorable items positively but less so if they used the Turnpike most of the time. On unfavorable statements about U.S. 1, agreement was consistent among drivers sampled on the Turnpike when questions were independent of the frequency with which the Turnpike was used. The U.S. 1 driver, however, had a definite shift from disagreement with unfavorable statements, if he were an infrequent user of the Turnpike, to a positive response if he were a frequent user.

The significant aspect (tables 10 and 11) is the fact that drivers sampled on the Turnpike made consistent responses to statements about both routes, whether they were frequent or infrequent users of the Turnpike. The drivers on U.S. 1, however, shifted significantly in response to both types of statements, according to whether they were frequent or infrequent users of the Turnpike, but the major shift was in response to the unfavorable type of statement. These responses were to items that seemed to be the most discriminating type in the scale. Accordingly, drivers sampled on the Turnpike showed significant stability in their responses, regardless of the frequency of their usage of the Turnpike. The drivers sampled on the Turnpike consistently agreed with positive statements about the Turnpike and disagreed with unfavorable statements. He also significantly agreed with statements about the unfavorable characteristics of U.S. 1. Drivers sampled on U.S. 1, however, showed an adaptability to change in their responses, which was a function of experience with the Turnpike. Conclusion from the foregoing analysis is that the negative characteristics experienced by drivers on U.S. 1 in relation to the Turnpike caused drivers to shift to the Turnpike and minimized the probability of Turnpike drivers shifting back to U.S. 1.

Speed Volume and Traveltime Results

On the Turnpike, speed and volume were determined on a sampling basis. Speed and volume measurements were made at 10-mile intervals, both northbound and southbound. A radar speed meter was mounted in the rear of a stationwagon that was parked on the shoulder. The speed meter was aimed at the approaching traffic at an angle of about 10°. This angle was larger than is recommended for the most accurate speed measurements, so some error is in these measurements. Normally, a sample of 100 vehicles was counted, and the time required for them to pass the counting station was also determined. Thus, it was possible not only to determine the speed distribution but also to estimate the hourly volume passing that point. The same procedure was followed on U.S. 1.

The cumulative speed distributions for the Turnpike are shown in figure 6—similar data on U.S. 1 are also included. Data were kept separate for the two directions in morning and afternoon sampling periods. The mean speed of these samples (Turnpike) was approximately 61.9 miles per hour, and the standard deviation was 9.1 miles per hour. The speed distribution is slightly negatively skewed. These speeds should be considered cautiously for, as has been shown by Shumate and Crowther (*10*), there is nonhomogeneity among spot speed samples. For U.S. 1, the cumulative speed distributions also are shown in figure 6. The mean of this sample was 43.7 miles per hour and the standard deviation was 10.3 miles per hour. This speed distribution is also negatively skewed but not so much as that for the Turnpike. The variability of speeds, from

Table 8.—Relative frequency of trips of drivers sampled on Maine Turnpike and U.S. 1

Frequency of trips	Male drivers on—		Female drivers on—	
	Maine Turnpike	U.S. 1	Maine Turnpike	U.S. 1
Year	*Percent*	*Percent*	*Percent*	*Percent*
Less than 1	4.7	7.3	12.4	13.6
1–11	44.1	53.4	42.6	47.3
More than 12	51.1	39.4	45.1	39.0

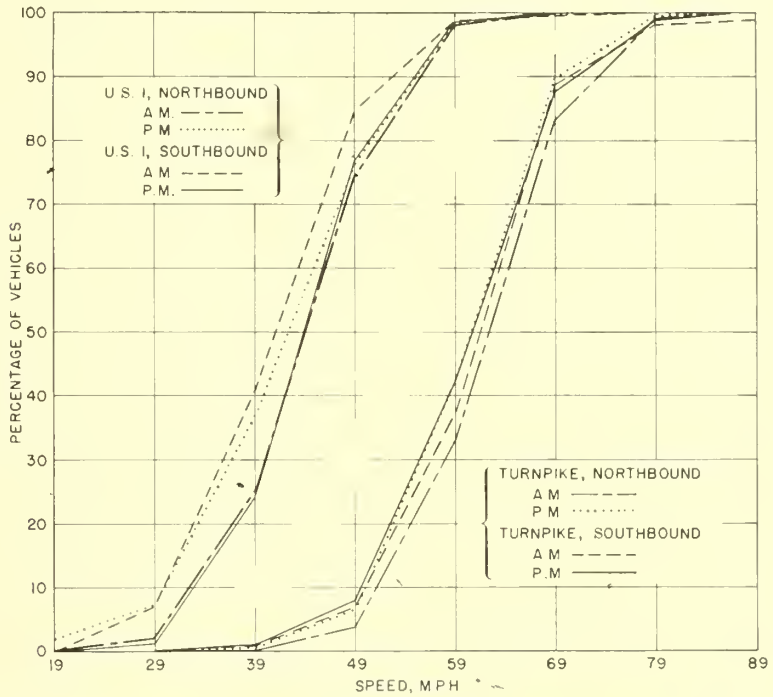

Figure 6.—Vehicle speeds on both routes, cumulative distribution.

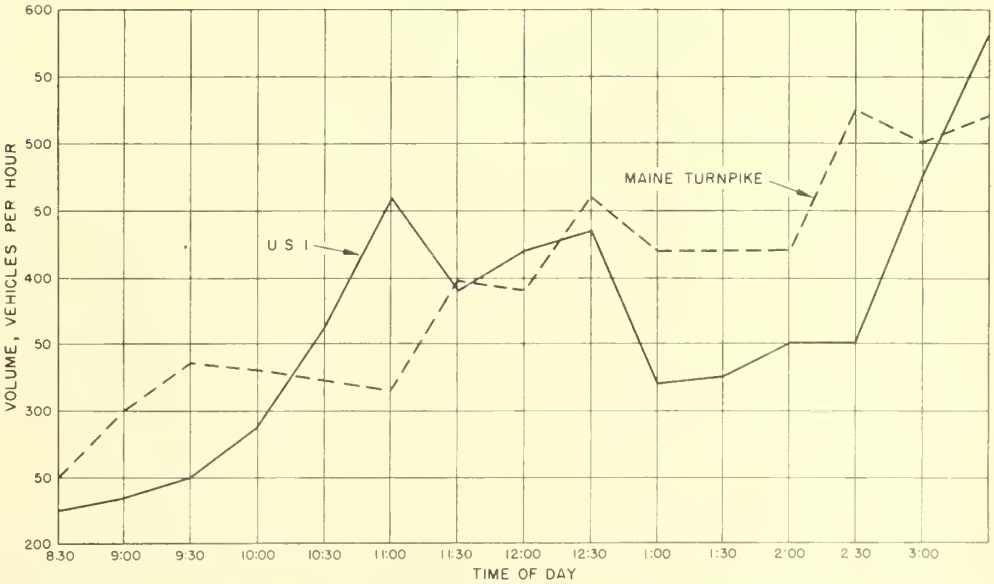

Figure 7.—Calculated average hourly volumes on both routes.

Table 9.—Mean attitudes toward the two highways as a function of the frequency of trips between South Portland and Kittery

Trip frequency, per year	Attitudes by age and sex of driver							
	Less than 24		24–34		35–44		More than 45	
	Male	Female	Male	Female	Male	Female	Male	Female
Maine Turnpike:								
Less than 1			40.07	39.00	41.18		36.93	
1–11	38.92	38.02	40.25	36.87	41.31	38.00	39.13	39.25
More than 12	43.21	33.78	43.23	40.33	42.93	41.10	41.77	38.24
U.S. 1:								
Less than 1	32.65	28.48	34.54	30.32	32.08	26.33	29.98	27.19
1–11	32.96	31.15	33.05	32.29	31.34	29.63	30.00	28.47
More than 12	31.32	31.54	34.97	32.79	33.68	29.12	30.72	30.36

Table 10.—Average item score of favorable statements for Maine Turnpike, by male drivers who use the Turnpike, either rarely or frequently

Percent drivers use Maine Turnpike	Favorable statements		Unfavorable statements	
	Maine Turnpike drivers	U.S. 1 drivers	Maine Turnpike drivers	U.S. 1 drivers
Less than 24	2.45	2.14	1.71	2.58
More than 75	2.54	2.44	1.70	1.70

Table 11.—Average item score of favorable statements for U.S. 1, by male drivers who use the Maine Turnpike, either rarely of frequently

Percent drivers use Maine Turnpike	Favorable statements		Unfavorable statements	
	Maine Turnpike drivers	U.S. 1 drivers	Maine Turnpike drivers	U.S. 1 drivers
Less than 25	2.09	2.60	2.46	1.61
More than 75	2.00	2.20	2.42	2.13

Table 12.—Traveltime between South Portland and Kittery on the Maine Turnpike and U.S. 1

	Maine Turnpike	U.S. 1
	Minutes	*Minutes*
Mean traveltime	41.1	63.9
Standard deviation	3.61	4.31
95 percent confidence interval	±1.25	±1.51

sample to sample and location to location, was much more on U.S. 1 than on the Turnpike. Therefore, the reliability of these summary statistics is questionable.

Volume of traffic was calculated for both the Turnpike and U.S. 1 on the basis of the same samples of the speed distribution. The average calculated hourly volume between the hours of 8 a.m. and 4 p.m. are shown for both routes in figure 7. The volume on U.S. 1 was not uniform over its entire 47-mile length; it was consistently larger at the more populous northern end. In addition, on U.S. 1, three counting stations were set up: One at each end of the study section and a third—a permanent counting station—about the middle of the test section. The calculated hourly volumes shown in figure 7 are approximately the same as those obtained at the counting stations. The volumes on the two routes were comparable and generally were parallel in their variations throughout the day.

Traveltime data were obtained from the trips made by the nine test drivers used for the GSR study. In these runs, the drivers were instructed to float with the traffic. This was done four times on each highway. Thus, 36 observations of traveltime were made on each route. Summary statistics are shown in table 10. The standard deviations indicate that on both routes the coefficient of variation in traveltime was 7 percent. This implies a variation for travel speed of approximately 17 percent on U.S. 1 and 14 percent on the Turnpike. Actually, the mean traveltime on U.S. 1 closely approximated the traveltime predicted from the mean speed of traffic on U.S. 1. On the Turnpike, however, the average speed of the test drivers was nearly 7½ miles per hour faster than that of traffic sampled on the Turnpike. This would indicate that the mean traveltime on the Turnpike for normal traffic may be up to 4½ minutes more than that shown in table 12. Finally, the maximum difference in time saved by selecting the Turnpike was calculated on the basis of the confidence intervals shown in table 12. In traveling between South Portland and Kittery a driver could obtain a maximum traveltime savings of 35 percent ±4 percent by driving on the Turnpike.

Tension Measurements

The data for the nine test subjects were analyzed by determining the peak magnitude of GSR for observed interferences that caused the driver to change his speed or position on the roadway. These interferences were: (1) Other vehicles traveling in the same direction, (2) vehicles merging into path of driver, (3) vehicles turning out of path of driver, (4) traffic control devices, (5) pedestrians on or near path of driver, (6) grades, (7) curves, (8) shoulder objects, and (9) opposing vehicles. The fourth—traffic control devices—appeared on the Turnpike runs as well as those on U.S. 1 because highway maintenance operations were continually performed on the Turnpike during the period in which GSR data were taken. Normally, advisory speed signs were placed on the highway to protect the maintenance crew, and these were included in the definition of traffic control.

The magnitude of GSR per minute, which is the defined measure of driver tension, was statistically analyzed by the analysis of variance. A summary of this analysis is shown in table 13. Significant differences were recorded between the routes and subjects but not direction. These results are similar to those reported previously (7). The comparison of tension between the two routes is shown for each subject in figure 8. The average tension differed considerably between subjects, but U.S. 1 generated significantly more tension for each driver than the Turnpike. The range of reduction of tension among this group of subjects on the Turnpike was from 22 to 61 percent. The overall average saving of tension by taking the Turnpike was 46 percent.

Each route was divided into four, 10½-mile sections. The tension data were analyzed to determine whether differences in tension were generated between the sections of the test routes. As had been expected, no significant variations from segment to segment were recorded on the Turnpike. Nor were significant differences recorded between the sections on U.S. 1. This was an unexpected finding because the highway and traffic from section to section of U.S. 1 had different characteristics and land use adjacent to the highway varied considerably. One reason for the lack of difference was that the predominant interference in generating GSR arose directly from other vehicles in the driver's path, rather than differences in sections of the highway. Furthermore, when driving through the more complex environments, all drivers reduced their speed and thus reduced the probability of unexpected interferences. These compensatory changes may well have eliminated any differences in GSR from the different sections.

Table 13.—Analysis of variance of GSR data

Source of variance	Sum of squares	Degree of freedom	Mean squares	*F*, ratio	Probability (*F*)
Routes	308.65	1	308.65	305.59	<0.01
Subjects	421.11	8	52.64	52.12	<0.01
Direction	0.89	1	0.89	0.89	(1)
Routes and subjects	62.60	8	7.83	7.75	<0.01
Routes and directions	28.59	1	28.59	28.31	<0.01
Subjects and directions	34.87	8	4.36	4.32	<0.01
Residual	44.38	44	1.01		
Total	901.14	71			

[1] Not significant.

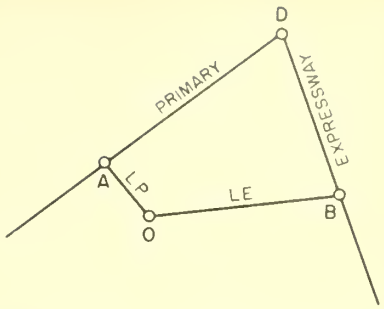

Figure 9.—Geometry of diversion situation.

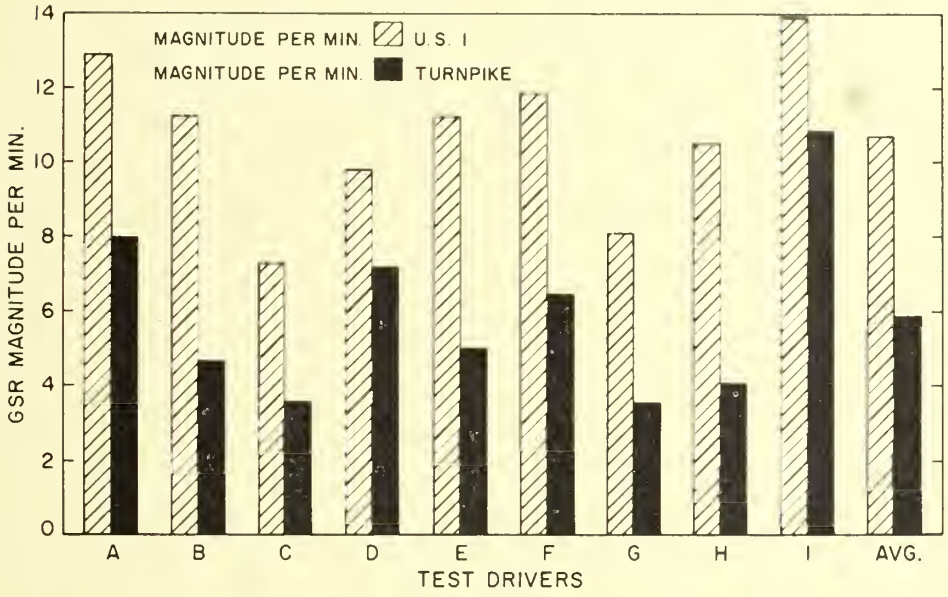

Figure 8.—Mean tension generation on both routes.

Interpretation of Results

One of the main objectives of this study was to determine whether drivers had stable attitudes that correlated their choices between alternate highways. The results clearly established that they do. The attitudes of the users of the one highway differed significantly from the attitudes of the users of the other. Furthermore, the users of the Turnpike had significantly positive attitudes toward that controlled-access highway, and users of the rural primary had significantly negative attitudes toward the Turnpike. On the basis of the results, only a small proportion of drivers who hold a positive attitude toward the Turnpike actually will drive on the primary. Furthermore, in the alternate choice situation studied, an attitude scale appears to be strongly related to choice, much more so than any descriptive information about the characteristics of the drivers or their trips.

The results of the study clearly showed that drivers do evaluate their experiences on different highways. This evaluation is developed from a variety of elements in the highways they travel. Whether consciously or unconsciously, drivers weigh the different features of highways and combine subjective experiences into an overall evaluation. This is reflected in attitudes and predisposes drivers toward the choice of one highway instead of another. As a matter of fact, it is these attitudes that overwhelm all the specific short-term aspects of a particular trip and dictate the choice of route.

A third aspect of the study concerned the problem of attraction of traffic to an expressway. In several of the analyses it was very evident that attitudes shifted toward favoring the Turnpike. The most clear-cut example is the one in which the individual items on the scale were analyzed according to the route sampled. The significant finding was that the more drivers use the two highways, the more the primary suffers by comparison. The learning experience apparently increases drivers' awareness of the negative characteristics of the primary, so they become more dissatisfied with it. The direct experiences obtained in driving the primary-type of highway seem to force drivers onto a turnpike. Thus, the overall problem of the attraction of traffic to an expressway may be considered to arise from the direct experiences drivers have in driving it and any alternate. Because the expressway is perceived by drivers to have fewer negative effects than an alternate primary, a slow shift to the expressway occurs that seems to be motivated by a desire to escape the characteristics of the highway of older design.

Three major factors inherent in this type of situation may motivate a shift in favor of an expressway. First is the reduction in travel-time obtained by choosing the expressway. However, the results of the study showed no significant shifts in attitudes as a function of driving time. Drivers have the same attitude about both routes whether they are traveling for one-fourth hour or more than 2 hours even though, as a proportion of the total trip, savings in time gained from taking the expressway are decreased for long trips.

Second, in the original validation study, an item relative to the time savings to be obtained on an expressway was nondiscriminating; that is, regardless of whether people have positive or negative attitudes toward a turnpike they all agreed that time could be saved on it. Thus, although all drivers knew there was a time saving, it had no influence on their attitudes. As drivers know this to start with, time savings cannot be the basic cause of the shift in attitudes favoring an expressway. Some more subtle aspect of driving must be the source and it seems to be most sensitive to the negative characteristics of the primary.

Third, the direct cost of travel to the user is a factor. However, this does not seem reasonable, as the shift is in the wrong direction. That is, if cost of travel were a significant determinant of choice, a shift of attitudes away from a turnpike would occur, especially as trip frequency increased. However, the results clearly showed that, as the frequency of trips increased, there was an increasingly positive attitude toward the Turnpike and an even more likelihood that a driver would choose it. Also, two items were added to the scale that directly affect economic evaluation by the driver. These two items were actually the same except that one dealt with direct out-of-pocket cost, whereas the other dealt with cost per vehicle-mile.

The two statements read, "I would always travel the Turnpike between South Portland and Kittery if the cost were no more than" and alternatives were provided; for example, one increased the cost from 25 cents to $4, doubling over each of the five categories and another increased the cost from one-half cent a mile to 8 cents a mile. As might have been expected, the cost per mile item was nondiscriminating. Very few drivers had any

idea of per mile cost. The result was that estimates on both routes were randomly distributed; a small proportion of drivers omitted a reply to the item. More surprising, actual out-of-pocket cost was also nondiscriminating. The reliability on the Turnpike was a little higher, possibly because the drivers had just received a toll ticket. Further, drivers sampled on both highways consistently reported to the interviewers that the cost of the Turnpike was irrelevant to their choice. This finding may simply mean that most drivers in this sampling were very indifferent to the expense of traveling the Turnpike at current cost levels.

Neither time savings nor direct costs seem to be dominant in determining the attraction of traffic to the turnpike. What seems to be required is something that drivers must learn by direct experience: Something related primarily to the negative characteristics of the rural primary type of highway. This leads inevitably to the consideration of the stresses arising in driving on the two routes. From the results of the GSR phase of the study discussed here, the tension aroused in the test drivers on the Turnpike was approximately one-half that generated on the primary. This tension was caused by interferences that had purely negative effects. It seems reasonable that shifts in traffic to an expressway facility is actually a forcing of drivers away from the primary route so that they can avoid its stress inducing characteristics. Stated more generally: Drivers make choices between routes to minimize the total stress to which they are subjected in driving. Thus, for the passenger car driver, the basis for scaling the benefits to be obtained from using an expressway are neither economic nor timesaving, but they are stress saving.

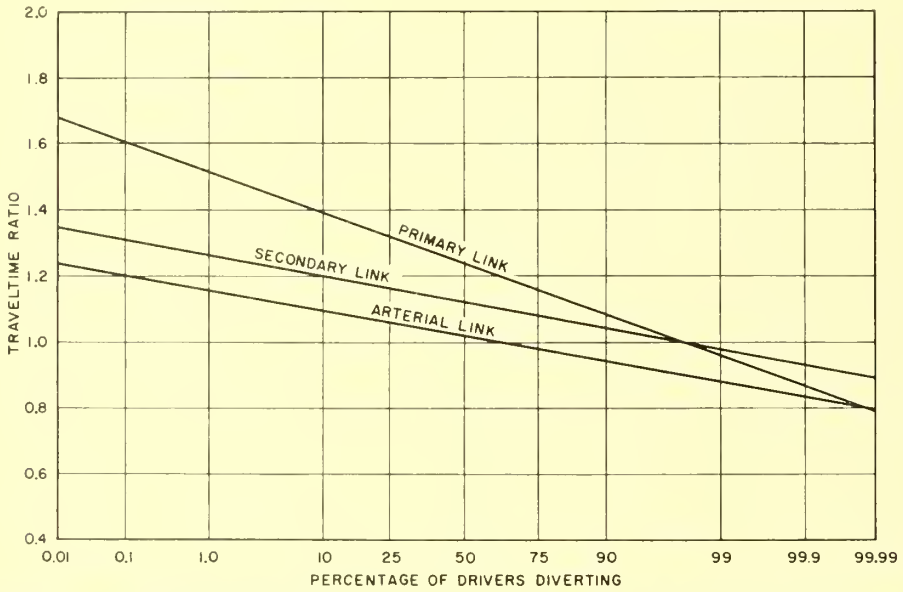

Figure 10.—Theoretical diversion distributions, different connections from primary to expressway.

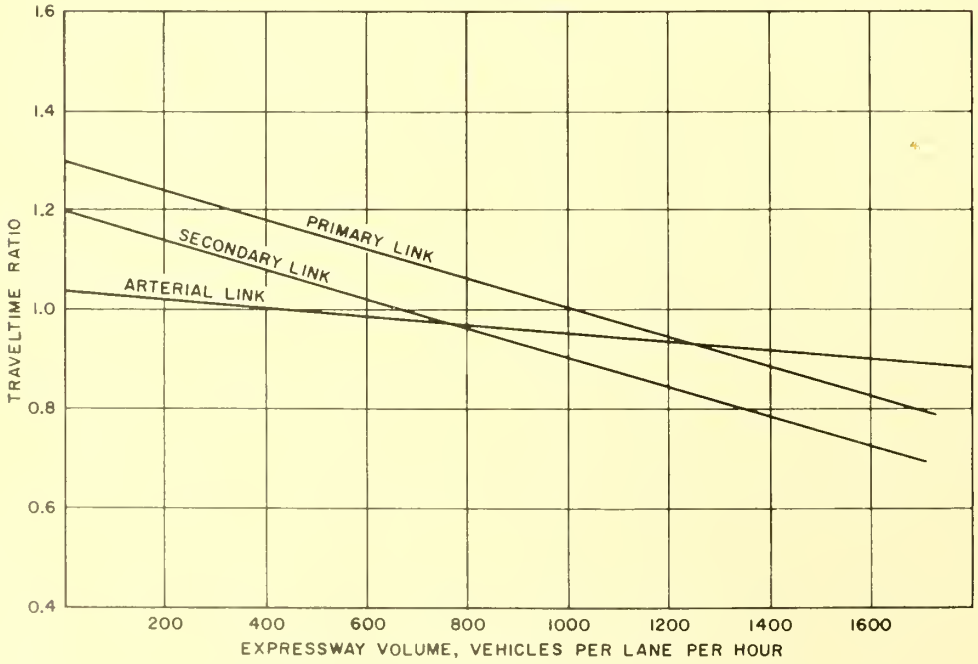

Figure 11.—Expected traveltime ratio, 50 percent diversion, as function of expressway volume.

The objective of minimizing the stress level in driving may explain two characteristics of the distribution of trips in the study results. First, the more frequent a trip, the more likely the drivers were to take the Turnpike. Second, the longer the duration of the trip, the more likely it was to be made on the Turnpike. Obviously, the total stress experienced on either route was a function of the particular properties of the route and the duration of the trip. That is, the total tension incurred is the integration of the unit stress over the duration of the trip. These tension inducing interferences occur randomly in time, the mean value being more on the primary highway than on the Turnpike. Because the variance in rate of occurrence of tension inducing interferences is high, the differences between the stress experienced on two highways in any short time interval will be unpredictable. Frequent repetitions or an increased sampling interval—that is, longer trips—will be required for the driver to reliably detect the difference between the alternates. By making frequent repetitions or longer trips, drivers will more likely detect the differences in tension on the alternate routes and thereby modify their choice behavior. The traveltime distribution and trip frequency data collected for this study conform to this hypothesis.

In simplest terms, the tension generated on any trip is some function of total traveltime and the frequency and intensity of stressing interferences. Using a relative measure of tension, a dimensionless constant is obtained. The relative stress obtained on any trip on a highway may be defined:

$$S=\frac{T_n}{T_R}(t) \qquad (1)$$

Where,

T_n=magnitude of GSR per minute on highway n.

T_R=magnitude of GSR per minute on reference highway.

t=trip duration.

Thus, if tension generated on a freeway is used as a reference, a numerical value of relative stress can be calculated when the type of highway on which travel is done and trip duration are known. In this and previous studies (*6*, *7*) it was shown that tension generated relative to the controlled-access highway was approximately 1.8 for a primary highway and 3.3 for an urban arterial. For a rural secondary highway having a low volume of traffic, the ratio probably is intermediate between these two, or about 2.5. Similarly, the relative stress for any set of routes may also be computed by summing the stress for the components and the minimum stress route determined.

Relative to the problem of diversion to an expressway, this model suggests that: Drivers will divert to an expressway if the total stress

experienced in reaching the expressway and on the expressway to the destination does not exceed that of the trip from origin to the alternate highway and on the alternate to the destination.

A general situation is shown in figure 9. Assume that an expressway, E, and a primary P, have a common terminus. Also, assume that the origin of a trip is located in the space bounded by the two routes so that there is a direct connection to either by link L. According to the hypothesis proposed herein, a driver will divert to the expressway to reach his destination if the total tension generated on the link, L_E, and the expressway, E, is equal to or less than the tension generated on the link, L_P, and the primary, P. When the origin lies on the primary and link L is a perpendicular connection to the expressway, E (fig. 9), then an inequality is obtained, as shown in equation (2), which defines the minimum separation between the primary and expressway for which 50-percent diversion will occur:

$$K_L \sin\theta + K_E \cos\theta \leq K_P \qquad (2)$$

The constants are the relative stress developed on each of the links. The solution of equation (2) is simply derived. Solving in terms of the cos θ, a quadratic equation is obtained, the real root of which is shown in equation (3):

$$\cos\theta = \frac{\left(\frac{T_P}{T_E \cdot V_P \cdot V_E}\right) + \left(\frac{T_L}{T_E \cdot V_L}\right) A}{\left(\frac{T_L}{T_E \cdot V_L}\right)^2 + \left(\frac{1}{V_E}\right)^2} \qquad (3)$$

Where,

$$A = \sqrt{\left(\frac{T_L}{T_E \cdot V_L}\right)^2 + \left(\frac{1}{V_E}\right)^2 - \left(\frac{T_P}{T_E \cdot V_P}\right)^2}$$

$\frac{T_P}{T_E}$ = ratio of stress developed on a primary highway to that developed on an expressway.

$\frac{T_L}{T_E}$ = ratio of stress developed on the link between primary and expressway.

V = mean speed in m.p.h. on appropriate highway.

It is further possible to define the travel distance ratio and the traveltime ratio. The equations are:

$$\frac{d_L + d_E}{d_P} = \sin\theta + \cos\theta \qquad (4)$$

Where,

d_L = distance on link.
d_E = distance on expressway.
d_P = distance on primary.

and,

$$\frac{t_L + t_E}{t_P} = \frac{V_P}{V_L} \sin\theta + \frac{V_P}{V_E} \cos\theta \qquad (5)$$

Where,

t = traveltime on each link.
V = mean travel speed on each link in m.p.h.

By using the values for relative stress for three different types of highways and the travel speeds, equations (3), (4), and (5), may be solved, and the results will be as shown in table 14. The mean traveltime ratio decreases consistently as the stress inducing characteristics of the link increases.

Two other aspects may be considered by using this model. One is the variance in tension. In this analysis the relative stress is treated as a constant, although it is, of course, a mean value. On the basis of the data collected in this study, the variance of this ratio was 0.42. Using this ratio, it is possible to calculate the percentage of drivers diverting to an expressway, using equations (3) and (5). Normit plots are shown in figure 10 for the three examples. The other aspect concerns the volumes of vehicles the highways are carrying. As has been stated previously (*7*), the mean tension on an expressway increases linearly to about 1,400 vehicles per lane per hour. Beyond that volume tension increases very rapidly. On urban arterials (*9*) volume seems to have relatively little overall effect on tension generation. For primary highways, however, no data are available on the effect of increasing volume. If it is assumed that the effect of volume on the primary highway is similar to that on arterials, it is obvious that diversion to an expressway will vary solely with volume on that type of highway. The effect of increasing expressway volume on the traveltime ratio for 50-percent diversion is shown for the three types of links in figure 11. These curves were derived from equations (3) and (5). In all three examples, the traveltime ratio for 50-percent diversion decreases until, as volumes exceed 1,000 vehicles per lane per hour on the expressway, an actual time savings must occur before half the traffic diverts.

Note that the diversion curves developed from this special example do not conform to those developed from origin and destination studies in this corridor (*1*). The model predicts much more attraction than actually occurred; this was caused partly by the assumptions about the connection between primary and expressway routes. The choice points are not very direct for drivers within the Maine Turnpike and U.S. 1 corridor. Furthermore, a significant proportion of trips in that corridor are very short. For this kind of traffic, essentially trapped on U.S. 1, diversion to the Turnpike would gain the driver no detectable reduction in stress and, hence, little diversion would be expected.

However, for corridor trips of more than 10 miles and north-south oriented, considerably more diversion should occur than is shown in the general diversion curves (fig. 11). In this respect, Carpenter (*2*) examined through trips between Wells and Saco and reported that 30 percent of them diverted to the Turnpike, even though the traveltime ratio was approximately 1.22. However, on the basis of the link characteristics, the tension ratio for the alternate routes may be calculated and is approximately 1.09. This yields expected diversion of approximately 35 percent of these trips.

A reasonable conclusion is that whenever the alternates available are equally stress inducing, drivers will always choose the route that takes the least time. Therefore, it is not surprising that most drivers, when questioned as to why they chose the route, commonly used traveltime as a response. Not only is total stress directly related to traveltime but also, many of the alternates available offer no significant stress reduction. Furthermore, such trips are often so short that stress differences are hardly detectable. It is evident from results of the study reported, however, that drivers will actually tolerate a time loss, as well as a distance loss, if the total stress to which they may be subjected is perceptibly reduced. On the basis of this model, measures that reduce stress should cause both increases in trip length and trip frequency. As driving is a stressful and energy consuming task, each driver has a tolerance or limit beyond which the subjective cost of driving becomes excessive. The satisfactions to be gained by a trip are less than the energy required to achieve it. If trips are predominantly goal oriented, the stress imposed on a driver becomes the equivalent of a cost, the value of which is determined in part by the desirability of the goal. Conversely, reduction of this subjective cost by the addition of improved highways not only makes any given trip easier, but also makes lower priority goals more attainable. Thus, new travel is generated.

It would seem that the value of these subjective costs of driving could be determined experimentally, either: (1) by subjective scaling of simulated trips, which is a variation of game theory techniques, or (2) by subjective evaluation of actual trips made under well-defined conditions. However, a significant problem would remain: The measurement of the value a driver places on the need to make the trip. It is the ease with which the highway transportation satisfies this need that is the measure of the subjective benefits of the highway transport system. It would seem, then, that methods exist for quantifying the subjective costs of travel but not for subjective benefits. One thing, however, becoming increasingly clear is that, although passenger car drivers make rational evaluations of transportation, their benefit-cost ratio appears to have little in common with the economic criteria normally used in highway transport.

Table 14.—Theoretical solution of expected diversion from a primary highway to an expressway

Link type	Separation between primary and expressway	Trip distance	Travel-time
	Radians	*Ratio*	*Ratio*
Primary	0.99	1.39	1.24
Secondary	.34	1.28	1.12
Arterial	.13	1.12	1.02

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NEW PUBLICATIONS

New Highway Map of the United States

The Bureau of Public Roads has recently published a new highway systems map of the United States showing the National System of Interstate and Defense Highways, the Federal-Aid Primary Highway System, and the U.S. Numbered Highway System. The eight-color map is printed on a single sheet, measuring 42- by 65-inches. The scale of the map is 1:3,168,000; that is, 1 inch equals 50 miles, and it is drawn on the Albers equal-area projection. The actual map compilation was made by the U.S. Geological Survey, with the cooperation of the Bureau of Public Roads and the State highway departments.

The new map may be ordered under the short title, *Federal-Aid Highways*, from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, at $1.50 per copy.

In addition to the three highway systems, the map also shows national forests, parks, and Indian reservations; all information represented is as of March 1, 1965. It should be noted that the map shows highway routes without regard to condition or completion, and many of the Interstate System routes are not yet built. Although the map will serve many useful purposes, it is not a touring or road-condition map.

For the 41,000-mile National System of Interstate and Defense Highways, commonly called the Interstate System, the locations of all routes are shown on the map, but only about one-half of the mileage is open to traffic at present. The System is scheduled for completion by 1972. The Federal-Aid Primary System totals about 227,000 miles (exclusive of the Interstate System); the majority of its routes are parts of the State highway systems. The U.S. Numbered System, 170,000 miles in extent, was devised by the American Association of State Highway Officials as a means for guiding travelers; it does not designate Federal-aid highways. However, most U.S. numbered routes are on the road systems eligible for Federal aid.

Illumination Variables in Visual Tasks of Drivers

By [1] **H. RICHARD BLACKWELL, Director, The Institute for Research in Vision, The Ohio State University; RICHARD N. SCHWAB,[2] Electrical Engineer, Office of Research and Development, U.S. BUREAU OF PUBLIC ROADS; and B.S. PRITCHARD,[3] formerly Research Associate at the Institute.**

The research reported in this article originally was conceived as a very limited study of the illumination levels needed for adequate performance of certain types of visual tasks that might be required of drivers. The authors originally planned to apply a general method, previously developed for visual tasks related to interior environments, to problems related to determining the illumination requirements for visual performance of driving tasks at night. However, the general method was not entirely satisfactory and new techniques for studying these problems had to be developed. The new techniques are explained.

Data developed from this study show that drivers experience many different degrees of difficulty in performing visual tasks that might be encountered in night driving—the degree of difficulty experienced being dependent to a large extent on the factors that influence the background luminance and the contrast of the task. A very comprehensive study of illumination and visibility variable would be required before any general understanding of the problems related to seeing while driving could be achieved, according to the authors. They note that the study reported in this article is not such a comprehensive work but that the results obtained should be useful for defining variables of interest for further research on highway lighting requirements. Some of the pitfalls that should be avoided in this further research are discussed.

On the basis of the data presented and the assumptions made, the authors estimate that 1.30 footcandles of illumination would be required for a driver to see a small black dog 200 feet away in the driving lane, and that 1.85 footcandles of illumination would be required for the driver to see a manikin of a young girl dressed in a long gray coat in the same location as the dog. An analysis of the data compiled suggests that contrast is more important than luminance in defining visual tasks.

Introduction

THE RESEARCH reported here is based primarily on the *visual task evaluator* (VTE) technique described by Blackwell (*1*) [4] in an earlier publication. In the work discussed in this article the technique has been extended from the earlier work on illumination levels required to perform certain types of visual tasks, occurring in interior environments, to those types of tasks that a driver might encounter in a street or roadway environment. The technique leads to an index of visibility based on the extent to which a practical visual task exceeds the borderline point between barely seeing the task and not seeing it at all. This borderline point is called the threshold of visibility, and the visual task may be that of seeing any object in the visual field that may be of interest to the observer when it is viewed against its normal background environment. An example might be seeing a pedestrian standing by the side of the road.

The degree to which the practical task exceeds the threshold point is measured by using the VTE to reduce the contrast that the object has with its background until the object is no longer distinguishable when viewed through the VTE. The amount that the contrast between any object and its background must be reduced to reach threshold may be used as an index of the extent to which that object exceeds the visibility threshold.

In the original use of the VTE technique, Blackwell used this measure of contrast reduction to define a value $\tilde{C}$ for each task studied. $\tilde{C}$ is defined as the physical contrast of a 4-minute, luminous, disk target having a visibility level equivalent to that of the task of interest—equivalence being specified as an equal amount of contrast reduction required to bring each task to its visibility threshold. The 4-minute disk target can be used, therefore, as a comparison standard, and the contrast ($\tilde{C}$) of this target can be used to determine the illumination level required for a selected level of task performance based on laboratory performance data (*2*). In the study reported here, the VTE defines illumination levels in terms of a performance criterion adopted as a standard by the Illuminating Engineering Society (*3*, *4*). Several special procedures were required in applying this method to the roadway environment; they are described herein and their validity established.

In addition, the VTE technique specified the relative visibility of a task under different roadway conditions, thereby allowing the examination of the effect of different aspects of illumination upon visibility. The particular aspects of the roadway illumination examined include the type of light source, the type of pavement, the spacing between light sources, the location of the task on the roadway, and the distance between the observer and the task. The relative visibility also has been related to background luminance and task contrast—the two physical parameters that determine task visibility within the roadway situation.

This study data showed a wide range in the degree of difficulty of different visual tasks that might be encountered on roadways at night. Indeed, different tasks require levels of illumination that range from moonlight to full daylight. The difficulty of a task depends, to a most significant extent, upon the factors influencing background luminance and task contrast, and these include all the factors that affect the amount of illumination striking a vertical object and its horizontal background. This implies that a very comprehensive study of illumination and visibility variables in roadway visual tasks is required before any

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[1] This article is based on research conducted under Ohio HPS-HPR 1(32), *A Study of Highway Lighting*, by the Transportation Engineering Center, Engineering Experiment Station, The Ohio State University under sponsorship of the Ohio Department of Highways, and in cooperation with the U.S. Bureau of Public Roads. The project also was supported by the Illuminating Engineering Research Institute. A complete technical presentation of this research is available in reference (*5*).

[2] Mr. Schwab was formerly Research Assistant at The Institute for Research in Vision.

[3] Now dead.

[4] References indicated by italic numbers in parentheses are listed on page 248.

great understanding of the problem of seeing while driving can be achieved. The research reported in this article does not represent such a comprehensive study. It should be useful however, in defining the variables of interest for a more comprehensive study.

The primary data were collected at one test site in Hendersonville, N.C., where lighting variables could be controlled and changed readily. Other test sites also were used and the results obtained were very similar. For this article, only data from the Hendersonville test site were used to derive average values of the illumination required for roadway tasks because these were the most complete data (*5*). The lighting at the test site was assumed to be reasonably representative of general practice.

Summary

Field tests were conducted on the visibility of a series of realistic objects located on a test roadway having lighting that could be changed. Visibility was assessed through the VTE technique. It was necessary to develop special techniques when applying the VTE to the study of roadway visual tasks. One technique involved the evaluation of the visual effect of disability glare. A special attachment for a photoelectric photometer was developed to do this, and the results were analyzed on the basis of laboratory visibility data. Another technique involved the use of a small part of the visual field when evaluating the state of visual adaptation. Physical measurements of the contrast in several roadway tasks were used to demonstrate that visual adaptation should be measured over a small part of the field next to the most visible detail of the object, rather than over a much larger area as previously had been used with the VTE procedure.

Visibility assessments were used first to evaluate the influence of such variables as objects, illuminants, viewing distance, location of object on the roadway, location of object to luminaires, luminaire spacing, and pavement material. Roadway tasks were concluded to vary grossly in difficulty and all the variables studied had important effects upon target visibility. The relative significance of the different roadway variables developed from the test data obtained should be estimated with caution, until a more complete, theoretical understanding of the causative factors involved has been obtained.

The data have also been used to determine the illumination needed to bring roadway visual tasks to a level of performance currently used in defining standards by the Illuminating Engineering Society. Average values required for visibility of objects at a distance of 200 feet were 1.30 footcandles to see a toy black dog and 1.85 footcandles to see a little girl manikin. Frequency graphs were prepared to illustrate the number of locations on the roadway providing this criterion level of task visibility for different levels of roadway illumination. In 99 percent of the locations on a lighted roadway, about 4 footcandles were required to see the dog and about 5 footcandles were required to see the manikin. Data were also prepared to illustrate the relative levels of illumination required to increase visibility to the criterion level at distances of more than 200 feet. When the distance was increased to 400 feet, an increase in illumination of about 2.5 times was required to see the dog and about 15 times to see the manikin. Care must be exercised in interpreting these illumination requirements. First, illumination levels depend critically upon the geometry involved. Hence, the illumination levels derived from the test data can refer only to roadway lighting installations of the same geometry. Second, many of the conditions encountered at the test site may not apply to real roadways. For example the pavement surfaces at the test site were unusually clean and unmarked. Third, the visibility criterion adopted by the IES for indoor tasks may not be applicable to roadway tasks. Analysis of the data, however, illustrates the value of studying the roadway visual problem by using the VTE technique. These tests produced useful information on the relative influence of different roadway variables and required illumination for selected objects at a selected level of visibility.

Equipment and Calibration

The basic instrument used for the tests discussed herein was the original laboratory model of the VTE, which is shown mounted on the table at the right in figure 1; a Pritchard photometer is mounted on the tripod at the left. The extra lens beside the photometer control box is the *disability glare lens*, which is described subsequently. A schematic optical diagram for the VTE is figure 2. An observer looking through the VTE sees an image of the real world beyond the objective lens centered in the photometric comparator cube. Surrounding this central circular image of the external world is a doughnut-shaped *annulus* of uniform luminance produced by a lamp within the instrument. This annulus luminance is adjusted to equal the external world by a neutral absorbing wedge, labelled *annulus wedge*. This same lamp also illuminates a variable contrast wedge that is used to reduce the contrast of the image of the external world by a superimposed uniform light veil over the entire image seen through the instrument. The effect is similar to having a fog between the observer and the object viewed. The variable contrast wedge is constructed so that, at any given point on it, the total of the light transmitted through it from the external world and the light reflected from the internal light source are approximately a constant. For calibration purposes, a mirror, *M*1, is inserted to block the beam from the external

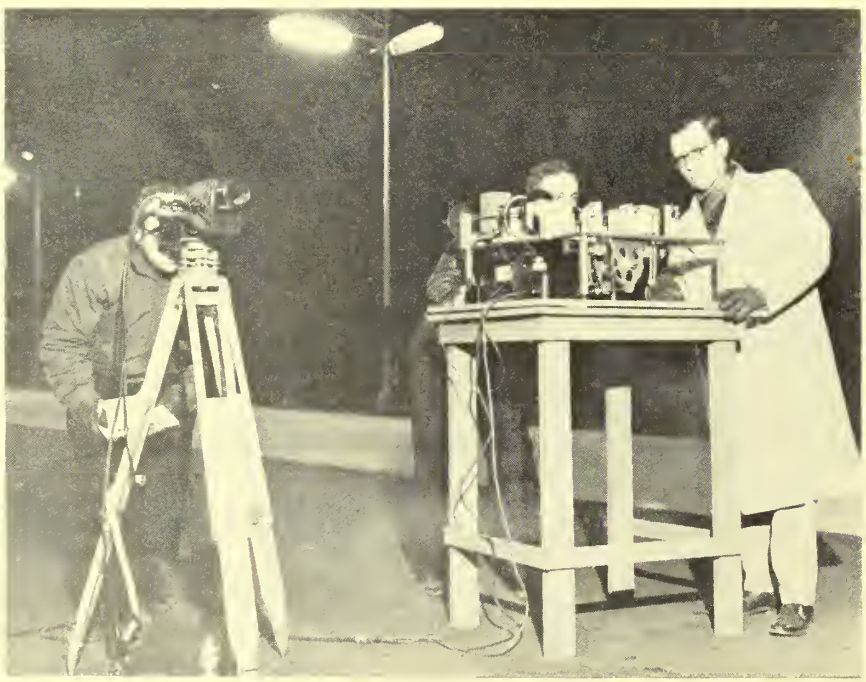

Figure 1.—Equipment for outdoor visibility test. Pritchard photometer, left; visual task evaluator, right.

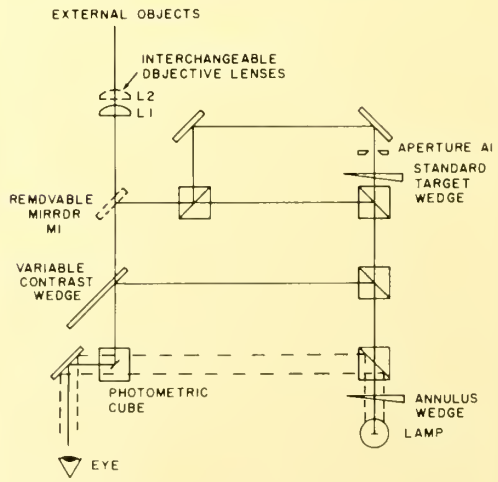

Figure 2.—Diagram of original visual task evaluator.

world and reflect a standard 4-minute disk target, the size of which is controlled by aperture $A1$; the contrast is controlled by the standard target wedge.

The calibration method used for the study reported here is summarized: In a photometric laboratory the transmission of the annulus wedge, T_A, was measured for all possible wedge settings. The luminance of an external object, when viewed through the VTE, was adjusted so that its luminance exactly matched that of the annulus when the annulus wedge was set for maximum transmittance. The luminance of the external object was then measured. This luminance, B_o, varied with lamp output and, therefore, had to be measured periodically. The next calibration determined the extent to which each setting of the variable contrast wedge reduced the contrast of the external scene. The extent of this reduction was termed *contrast rendition*, CR, and was measured by setting up an external object of equal luminance to the annular field when the variable contrast wedge was set for maximum transmittance. The transmittance, T, and the reflectance, R, were then measured photoelectrically by successively blocking the reflected and transmitted beam at different settings of the variable contrast wedge. The contrast rendition was defined as:

$$CR = \frac{T}{T+R} \qquad (1)$$

The final calibration measured threshold contrast for the standard target at several settings of the annulus wedge; this determined the background luminance against which the standard target was seen. With the removable mirror, $M1$, in place, the contrast of the standard target was varied by adjusting the standard target wedge until the 4-minute disk target was at the threshold of visibility. This process was repeated several times at each of several settings of background luminance by either reducing the contrast so that a visible target became invisible or by increasing the contrast of an invisible target until it became visible. Values of threshold contrast, C_m, thus obtained were plotted for different background luminances, and the smooth curve shown in figure 3 was drawn through them.

The values of C_m used represent the average of three sets of calibration data obtained by Pritchard between the fall of 1958 and the spring of 1960 and the original calibration data obtained during the 1957–58 VTE work on interior tasks. It was originally believed that data should be analyzed in terms of calibration data obtained at the same time each practical task was measured (*6*). Because it was subsequently learned that, for a practiced operator, use of an average calibration curve was preferable, average calibration data were used in the study discussed here. Also, for a second, untrained observer, this average calibration curve seemed to apply very well. In fact, when two observers attempted to make readings on the same practical tasks, the average calibration data (fig. 3) applied more reasonably to information obtained by the second observer than the calibration curves obtained directly by him. Therefore, the Pritchard calibration data were used in analyzing all VTE measurements, regardless of the observer.

Field Procedures

The original procedure for use of the VTE consisted of the following steps. First, the operator viewed the practical visual task through the VTE and centered the image of the task in the field of view. The variable contrast wedge was then set for maximum transmittance, the objective lens, $L2$, was inserted, and lens $L1$ was removed. Lens $L2$ produced a completely out-of-focus image of the external world, subtending exactly 2 degrees of visual angle. The resultant blurring of the external world image integrates the luminances within the field of view and produces the appearance of uniform brightness over the central circular area of the photometric cube. The brightness of the surrounding annulus, controlled by the annulus wedge, was then easily adjusted to match the average brightness of the central area. The average luminance, $\overline{B}$, of the task was defined from the calibration described earlier as:

$$\overline{B} = B_O \times T_A \qquad (2)$$

The annulus wedge was left in the position of the photometric match. Objective lens $L1$ was substituted for $L2$ to form an in-focus image of the external world. The variable contrast wedge was adjusted until the visual task of interest was reduced to threshold visibility and the contrast rendition, CR, read for that setting of the variable contrast wedge. The equivalent contrast, $\tilde{C}$, was then defined as,

$$\tilde{C} = \frac{C_m}{CR} \qquad (3)$$

where, C_m is the value read from figure 3 at a background luminance equal to $\overline{B}$. $\tilde{C}$ measures the intrinsic visual difficulty of the task because of physical variables such as object size, shape, luminance contrast, and chromatic contrast. $\tilde{C}$ does not reflect the difficulty of the task related to the background luminance present because its use in establishing the illumination requirements of different visual tasks requires $\tilde{C}$ to be independent of the illumination level present at the time the visual task is assessed.

After a value $\tilde{C}$ for a task has been obtained, the background luminance, B_r, that is required for performance of the task at a selected level of adequacy can be determined. As mentioned in the introduction, the performance criterion used in this article was based on certain assumptions of what constitutes adequate performance. These assumptions were: (1) That the task—detecting the presence of the standard object—be performed 99 percent accurately by trained laboratory observers; and (2) that information about the task be derived at the rate of five assimilations per second. To compensate for the difference between use of laboratory observers and so-called *commonsense seeing* and other variables, such as lack of complete information as to where and when the object was to appear, a field factor of 15 was introduced to adjust the laboratory performance data upwards. Justification for using these assumptions has been discussed by Blackwell (*1*, *4*).

Based on the preceding assumptions, laboratory performance data can be obtained to relate contrast threshold to background luminance, B_r, required to reach a certain performance level. Such a curve is shown as the solid line in figure 4. The ordinate corresponds to the logarithm of $\tilde{C}$ and the abscissa to the logarithm of B_r; therefore, after $\tilde{C}$ was measured, B_r was obtained by reading the curve in figure 4

The required illumination, E_r, was computed from the value of B_r. In the roadway study, the relationships were solved:

$$E_r = B_r \times \frac{\overline{E}_h}{\overline{B}} \qquad (4)$$

Where,

$\overline{E}_h$ = the average horizontal illumination provided by the roadway lighting system.

$\overline{B}$ = the average luminance of the task as defined in equation (2).

The logic of equation (4) is explained in the rest of this paragraph. The roadway lighting system producing average illumination, $\overline{E}_r$, provides luminance $\overline{B}$ for a particular task at some point along the roadway. If the visual task assessment showed that a luminance, B_r, was required to perform the task at the selected level of adequacy, the ratio $B_r \div \overline{B}$ represents the extent to which the lighting system produced an adequate luminance. *Assuming no change in illumination geometry*, the required average illumination, E_r, would equal the actual average illumination, times the ratio $B_r \div \overline{B}$. It cannot be overemphasized that no change in illumination geometry must be assumed. Obviously, in a three-dimensional situation such as in roadway lighting and viewing, unless the illumination geometry is maintained precisely, a change in illumination level could alter task contrast and, hence, task visibility. The assumption used in writing equation (4) is that, in effect, the system of roadway illumination is on a dimming control. The illumination could, therefore, be set at E_r to provide a selected level of visual performance for any task of interest by adjustment of the illumination up or down to the required level.

Disability Glare

In order to apply the VTE technique to a roadway environment, a special method was employed to allow for the deleterious effects of disability glare on task visibility. The field of view of the VTE was limited to the central 2-degree area around the object.

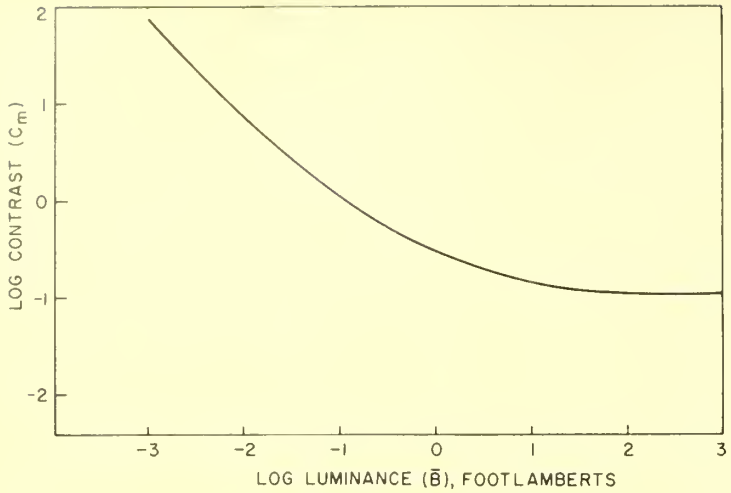

Figure 3.—Variation in threshold contrast as a function of background luminance.

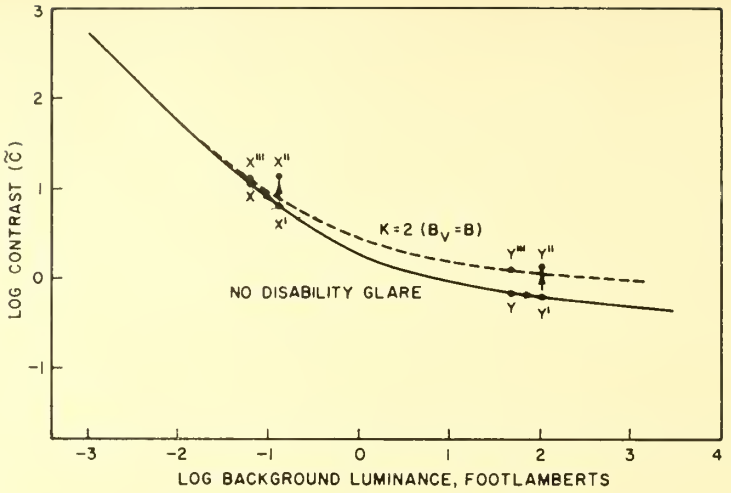

Figure 4.—Background luminance as a function of target contrast for standard level of visual performance: No disability glare solid curve; disability glare, dashed curve.

Because the main sources of disability glare were the luminaires located outside this area, these effects were not included in the initial visibility assessment. It might have been possible to enlarge the area viewed through the VTE by changing lenses; however, because of the physiological differences of individual observer's reactions to glare, it seemed preferable to use a calculation method.

The method used depended on the effects of disability glare described in an earlier publication by Blackwell (*2*). Disability glare can be assessed in terms of a uniform luminance veil, B_v, that is superimposed over the entire field of view and is equivalent in its effect on visibility to all the discrete sources of luminance in the field. The effects of disability glare are shown in terms of the standard performance curve in figure 4. The value of veiling luminance, B_v, that is equal to the disability glare effect increases the direct luminance, B, to B_e, the effective luminance, where:

$$B_e = B + B_v \qquad (5)$$

The increase in luminance produced by disability glare is shown for two initial values of B, designated X and Y. The task contrasts required for the standard level of performance are indicated by the location of the points X and Y on the solid curve. At the corresponding value of B_e, the contrast required for the eye to see at a selected level of visual performance is decreased by an amount equal to the differences between points X' and Y' and the original points X and Y.

Disability glare has a second effect, that of reducing the task contrast present; this effect may be described as:

$$C' = C \times \frac{B}{B + B_v} \qquad (6)$$

Where,

C' = the apparent task contrast in the presence of the disability glare, B_v.

C = the initial contrast of the task.

Because disability glare decreases task contrast, the physical value of task contrast must be increased to provide the contrast needed for adequate performance. This effect is shown in figure 4 by a comparison of the location of the points X'' and Y'' with those of points X' and Y'. The horizontal displacements of X' from X and Y' from Y are precisely the same on a double logarithmic plot as the vertical displacements of X'' from X' and Y'' from Y', equations (5) and (6). The values of X'' and Y'' are the contrasts required at the luminance values B rather than the values B_e, so they must be plotted at the locations X''' and Y'''. The constructions used in locating the points X''' and Y''' may be used for all points falling on the standard performance curve. The dashed curve (fig. 4) represents the resultant effect of disability glare on the standard performance curve when B_v is equal to B. A disability glare constant, K, was used to define the amount of glare present as:

$$K = \frac{B_v + B}{B} \qquad (7)$$

In figure 4, B_v was assumed to be equal to B, so K is equal to 2.

The method for determining the value of B_r in the presence of disability glare requires use of the dashed curve in figure 4, rather than the solid curve. Obviously, for a specified ordinate value of $\tilde{C}$, the luminance required to attain a specific level of performance is higher when disability glare is present than when it is absent. For convenience, the background luminance required when disability glare is present is referred to by the notation B_r'. Similarly, E_r' is used to refer to the required illumination in the presence of disability glare. For a fixed value of K, the larger the value B_r and E_r were originally, the more B_r' will exceed B_r and E_r' will exceed E_r.

To compute the values of E_r', a measure of the value of B_v in each roadway situation was required. Individual values of the illumination produced at the eye by each glare source could have been measured for each situation and a value for B_v computed; however, the work for this type of approach seemed prohibitive. A photometric device for direct measurement of B_v was required.

Some years ago, Fry (*7*) described a device consisting of a wide-angle lens that forms an image of the entire world out to 90 degrees on either side of straight ahead and an absorbing photographic mask that selectively transmits illumination coming from different points in the field in different proportions to satisfy an empirical formulation for disability glare. Such a device could be utilized as the objective lens of a photometer so that the summation could be performed photometrically. The device, although simple in principle, was exceedingly difficult to construct. The image produced by the wide-angle lens was distorted,

Figure 5.—Night view of three targets at test site.

and the photographic mask, therefore, had to have the same spatial distortion built into it. Furthermore, the transmission of light through the mask was required to change over a range of from 10,000 to 1. It was not possible to achieve this range in one piece of photographic material. Therefore, two separate masks were required, each having a central opaque spot that excluded all light from the central 2-degree area and symmetrically graduated density that radiated out from the central spot. An improved design for a disability glare lens has been described by Fry, Pritchard, and Blackwell (*8*).

In actual use, the Pritchard photometer was pointed at a visual task of interest and a value of average task luminance was obtained. The ordinary objective lens was then removed, and the disability glare lens was substituted for it, without moving the photometer. A photometric reading was made using each of the two masks. The effective luminances obtained were added to equal B_v. The value of K was computed from equation (7). After computing the value of K, allowance was made for the 7-percent component of disability glare when the eye was exposed to a field of uniform luminance, as shown by Moon and Spencer (*9*). The visual performance data represented by the solid curve in figure 4 contain this magnitude of disability glare. Thus a value of K' was computed from the relation:

$$K' = \frac{B_v + B}{B} - 0.07$$

$$= K - 0.07 \qquad (8)$$

The value of K' was used to construct contours such as the dashed curve (fig. 4), because the solid curve represents a baseline with the 7-percent disability glare already present. These procedures suffice for the computation of values of E_r and E_r' in practical roadway situations.

Relative Visibility Calculations

To arrive at an understanding of how different illumination variables affect visibility, it was necessary to obtain a measure of the relative visibility of a specific task under different conditions. Such a measure is the relative visibility factor (RVF), which is defined as:

$$RVF = \frac{\tilde{C}}{\overline{C}} \qquad (9)$$

Where,

$\tilde{C}$ = the equivalent contrast of the standard target.

$\overline{C}$ = the value of target contrast for the standard level of visual performance at the luminance $\overline{B}$ (solid curve in fig. 4).

The value of RVF is an indication of the difficulty of the visual task in terms of object size and shape, luminance and chromatic contrast, and average task luminance. RVF thus differs from $\tilde{C}$ only in the significance of the absolute values of the two quantities and in the fact that it reflects the effect of the level of background luminance, whereas $\tilde{C}$ does not. A value of RVF equal to unity signifies that the roadway illumination provides exactly the level of visual performance represented by the standard performance curve. RVF values larger than unity show that the task is more visible than required to meet this performance criterion, whereas RVF values less than unity show that the task is not as visible as is required.

As for the required illumination values, allowance for disability glare can be accomplished by adjusting the standard performance curve. The relative visibility factor in the presence of glare (RVF') is defined as:

$$RVF' = \frac{\tilde{C}}{\overline{C}'} \qquad (10)$$

Where,

$\overline{C}'$ = the value of $\overline{C}$ adjusted for disability glare.

Visual Tasks

It was believed desirable to utilize realistic roadway tasks that might be fairly representative of collision type situations rather than simplified tasks such as black disks that frequently have been used in similar studies. For primary use, a toy black dog and a manikin of a 12-year-old girl were selected. The manikin was outfitted in a loose-fitting, full-length gray coat having 20-percent reflectance. In addition, one series of measurements was made on seven other objects: (1) Black disk, 1 foot in diameter; (2) manikin wearing a coat having 60-percent reflectance; (3) toy, pink poodle dog; (4) black automobile without lights or retro-reflectors; (5) yellow highway cone marker; (6) bicycle lying flat on the roadway; and (7) red brick. The manikin, in the coat having 20-percent reflectance, and the two dogs are shown in figure 5, at the test site.

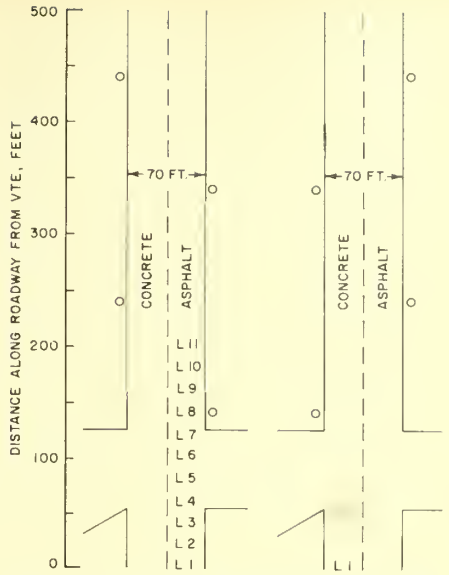

Figure 6.—Plan of layout at test site.

Roadway Installations

The plan layout of the test facility is shown in figure 6. The right half of the roadway was paved with asphalt and the left with concrete. The surrounding ground sloped off toward the right and toward the far end of the road. The area was wooded, particularly toward the right side. A white frame house was in the woods at the far end of the street. Luminaire poles were spaced at 100-foot intervals on each side of the roadway, as illustrated. Five poles were used for the first two series of tests; for the third series, a sixth pole was added at the end of the roadway, 230 feet beyond the last luminaire on the left side. Each pole had a 4-lamp fluorescent luminaire mounted transverse to the curb, a 400-watt mercury lamp, and an incandescent luminaire that could accommodate either 6,000- or 15,000-lumen lamps. Only one type of luminaire was used on a specified series of measurements.

The VTE was set up in the middle of the driving lane on the appropriate side of the pavement to be used in the particular series of measurements, as shown in the elevation layout in figure 7. The first operating luminaire, shown as a small circle (fig. 6), was always on the same side of the roadway as the measurement booth. The luminaires were spaced 200 feet apart on each side of the roadway, in staggered locations. In one test, only half the luminaires were used, and the spacing between them on one side of the road was 400 feet. Because the basic arrangement of luminaires was staggered, the spacing for this one test was designated as 200 feet.

Experimental Data

Three series of tests were made at Hendersonville, N.C. For convenience, these tests have been designated as test series I, II, or III. Each series is described separately because several important changes in the experimental technique were made as the work proceeded. The results have been analyzed for all three series of measurements and are included in the analysis of data.

Test series I

In the first series of measurements made at Hendersonville in the spring of 1959, the visual task, type of light source, type of pavement, spacing between luminaires, and lane in which the task was located were varied in a systematic way. After the data had been analyzed, certain questions arose as to the validity of the method previously described for using the VTE. It seemed from the data that the tasks became more visible as the viewing distance was increased. This effect was opposite to that expected on the basis of the object's decreasing angular size. The annulus brightness had been matched to the average brightness of an out-of-focus image, thereby equating the average luminance of the field of view to the luminance of the internal light source. It was suggested that the method used might have distorted the experimental data and produced these unexpected results. This would have been true if the eye had been adapted to the brightness

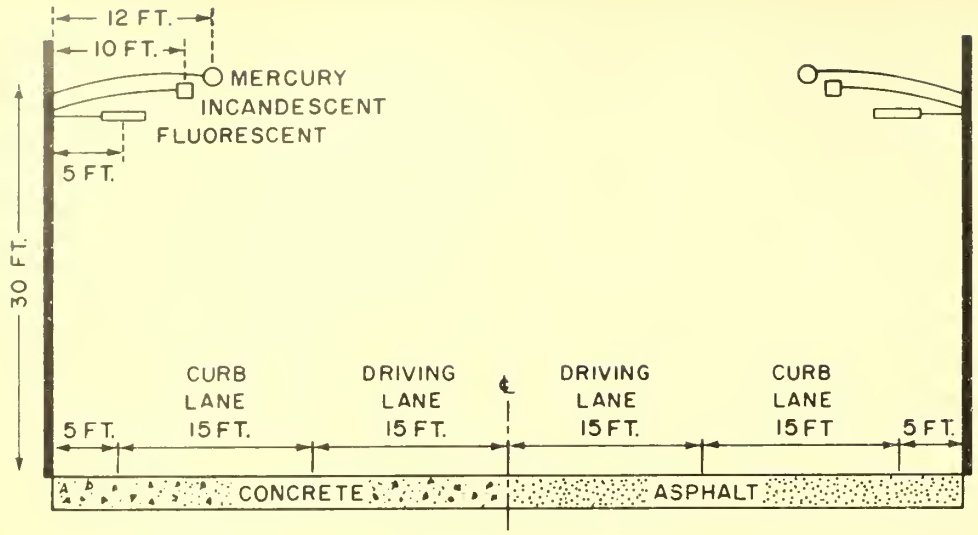

Figure 7.—Elevation layout of Hendersonville test site.

at a point in the visual field near the object rather than to an average field brightness. The second series of measurements were designed to investigate whether the procedure for measuring field brightness had introduced an error into the results.

Test series II

Two approaches were used in the investigation of the VTE procedure. First, a new procedure was developed that would be free of the suspected error. On the basis of earlier work (*2*), measurements were made of the physical luminances expected to influence visibility under the different conditions. Then predictions were made as to the relative visibility of the several objects viewed at different distances, and the two VTE procedures were used to make measurements of these objects. These measurements were analyzed in relation to the predicted visibility values. Most of the variables studied in test series I also were employed in use of the new procedure. An observation distance of 180 feet was employed.

The new VTE procedure, designated the *final* procedure, included several steps: Before setting the annulus wedge carefully, the variable contrast wedge was adjusted to ascertain the part of the object that disappeared last and, hence, was initially most visible. The VTE then was directed so that the background adjacent to the most visible part of the object was at the edge of the circular inner field of the photometric comparator in juxtaposition to the annular field. Objective lens *L1* was left in place so that the image of the external world was in focus. Then, the annulus wedge was set to match the brightness of the selected area of the background, and the variable contrast wedge was set for maximum transmittance. From this point on, the procedure followed was exactly the same as in test series I. Because the *blurring lens* was not used in the *final* procedure and because of the resultant nonuniformity of the background luminance, it was somewhat difficult to obtain a photometric match between the small part of the background and the surrounding annular field. Otherwise, the procedure for test series II caused no difficulties.

Physical measurements were made of the luminance of the most visible part of the object and its adjacent background, as determined by the *final* VTE procedure. The Pritchard photometer was used; its aperture restricted the field to a diameter of 10 minutes of circular arc. Photographs were taken of the target under each different condition, so the visual area of the relevant element of the target could be computed with precision. Comparison of the values of $\tilde{C}$ obtained from the two VTE procedures showed equivalent results for all targets at distances of less than 220 feet. At longer distances, the value of $\tilde{C}$ obtained from the original procedure was substantially larger than the values obtained from the *final* procedure.

The relations of *RVF*, *RVF'*, and $\tilde{C}$ were judged from the data shown in figure 8; the ordinate scale on the left of the figure shows the values of *RVF* and *RVF'* and the ordinate scale on the right shows the values of $\tilde{C}$. The role of disability glare at different locations in the roadway installation can be ascertained

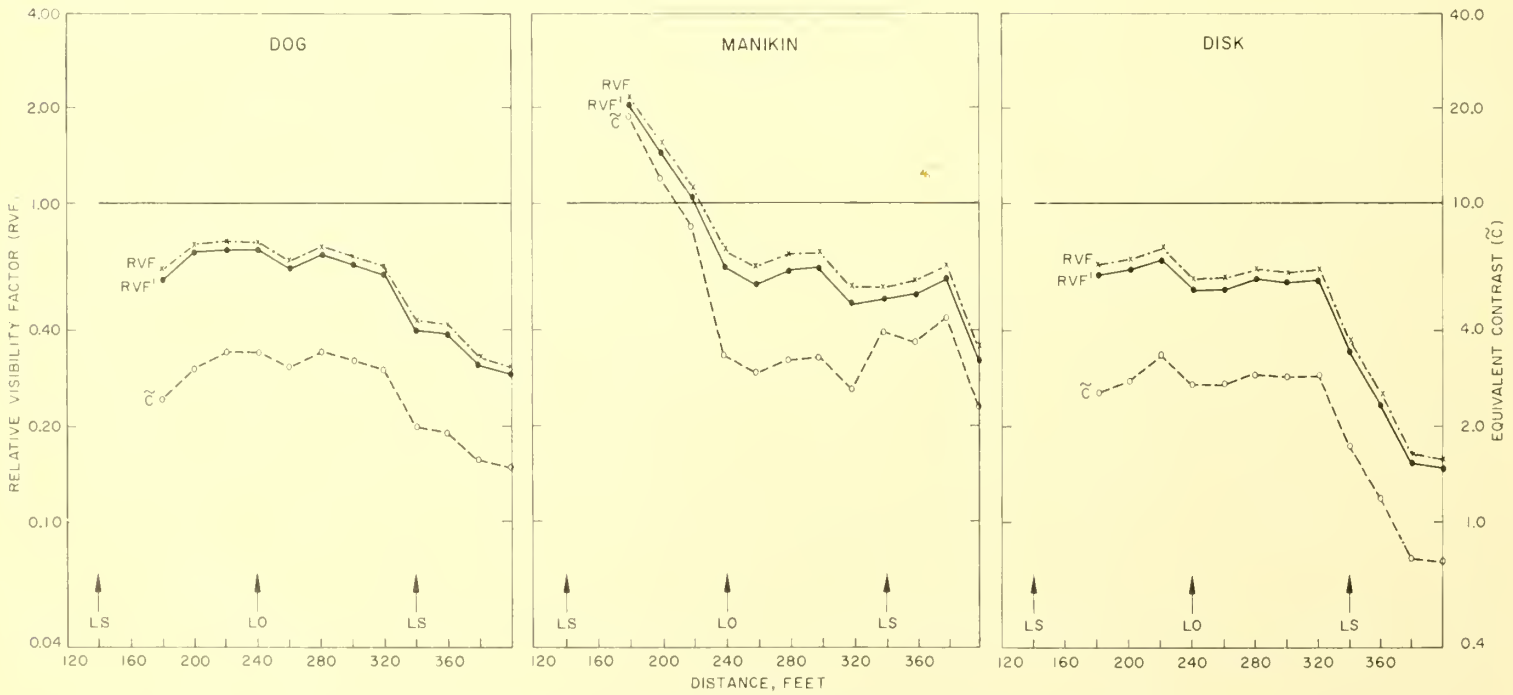

Figure 8.—RVF, RVF', and $\tilde{C}$ as functions of visibility distance in relation to location of luminaires. Horizontal line represents level of visibility required for standard performance level.

by comparing the values of RVF and RVF'. As stated previously, the role of variations in pavement luminance may be evaluated by comparing values of $\tilde{C}$ and RVF. In figure 8, the luminaire locations are indicated by: *LO*, luminaire on the opposite side of the roadway, and *LS*, luminaire on the same side of the roadway as the task. Values of RVF, RVF' and $\tilde{C}$ change according to distance in much the same way (they are parallel), thus establishing that these variations in background luminance as a function of distance were not important causative factors in determining object visibility at different distances. In almost no test did the values of RVF' exceed unity. Therefore, the lighting system was not producing a level of visibility sufficient to satisfy the performance criterion.

Test series III

The third series of test measurements were made because of a desire to obtain additional data under the VTE procedure used in test series II. In particular, it seemed desirable to study the relationships between visibility indices and distance for illumination geometries other than those obtained in the earlier measurements, in which the VTE was always located at position *L*1, as shown in figure 6. During test series III, the VTE was located at each of the 11 positions, *L*1 to *L*11. At each position, the dog and manikin were moved so that the distance between the object and the observer ranged from 180 to 400 feet. All these measurements were made on asphalt pavement, under 15,000-lumen incandescent luminaires at 100-foot spacings, and the objects were located in the driving lane.

Analysis of Data

The focal point of interest for the test series II was to test the extent to which the original and *final* VTE procedures yielded visibility indices in agreement with expectations based upon physical measurements. The luminances of the most visible detail of each object and its background for each of several distances were measured. These data were used to compute a measure of the target visibility expected to exist. The procedure involved the following described steps: The luminance contrast was computed from the relation proposed earlier by Blackwell (*10*):

$$C=\frac{B_t-B_b}{B_b} \qquad (11)$$

Where,

B_t=object luminance.
B_b=background luminance.

Then the contrast was adjusted by a factor to allow for the fact that the area of the object differed under different conditions. The factor F was defined as:

$$F=\frac{\overline{C}_s}{\overline{C}_a} \qquad (12)$$

Where,

$\overline{C}_s$=threshold contrast for a 4-minute luminous disk.
$\overline{C}_a$=threshold contrast for a target having the angular size of the element of greatest visibility.

Values of $\overline{C}_s$ and $\overline{C}_a$ were read for the particular background luminance, B_b, from the visual threshold curves of Blackwell (*2*) for 1-second exposure duration. These threshold data are for circular objects. In making these calculations, noncircular elements were considered to have the same threshold contrast as circular objects of equal area. A value of contrast obtained in equation (11) was then adjusted by using equation (12) to allow for differences in target size as:

$$C'=CF \qquad (13)$$

Generally, good agreement was obtained between the physically measured value of B_b obtained with the Pritchard photometer and the value of $\overline{B}$ obtained from the annulus wedge settings on the VTE. However, there were tests in which the two values disagreed considerably. This was particularly true of the data obtained under the *original* VTE procedure. It seemed more reasonable to conclude that the value of $\overline{B}$ was in error because of the comparative difficulty and uncertainty in visual photometric measurements. Errors in $\overline{B}$ would be expected to alter values of $\tilde{C}$ as related to C'. When $\overline{B}$ was too large, $\tilde{C}$ would be reduced because the veiling luminance would be larger than it should be. Conversely, when $\overline{B}$ was too small, $\tilde{C}$ would be spuriously large. A correction factor F' was developed where:

$$F'=\frac{\overline{C}_{\overline{B}}}{\overline{C}_{B_b}} \qquad (14)$$

Where,

$\overline{C}_{B_b}$=threshold contrast for an object having the area of most visibility at B_b.
$\overline{C}_{\overline{B}}$ =threshold contrast for the same element at $\overline{B}$.

These threshold values were also read from the same threshold curves used for equation (12). Then the corrected computed equivalent contrast of a traget element was determined:

$$C''=C'\ F' \qquad (15)$$

The correction factor F' reduced C' whenever $\tilde{C}$ was spuriously small, or increased C' whenever $\tilde{C}$ was too large. Thus, in effect the correction was being made in the wrong quantity. This should be remembered when considering values of $\tilde{C}$ as related to C''.

Values of $\tilde{C}$ obtained under the original and *final* VTE procedures were than evaluated. These values were compared with corresponding values of C''. Data for various objects under different luminaires and pavement combinations are presented in Part A of figure 9 for the original and Part B for the *final* procedure. Double logarithmic plots are used. All of these data represent a fixed distance of 180 feet. Thus, there is no parameter along which to order values of C'' and, hence, figure 9 contains only a simple regression line. The solid line in each part of the figure has a 45-degree slope representing that $\tilde{C}$ is proportional to C''. Because the line does not pass through the (0, 0) origin, $\tilde{C}$ is proportional to a constant times C''. This is, of course, acceptable because there was no satisfactory way to relate the threshold data and the measurements made with a VTE. The data seems to cluster more closely about the regression line in Part B than in Part A, particularly the data for the manikin. This was interpreted to mean that the values of $\tilde{C}$ obtained under the *final* VTE procedure agree more closely with the computed indices of visibility than do corresponding data obtained under the original procedure.

A better procedure for evaluating the values of $\tilde{C}$ obtained at various distances in terms of corresponding values of C'' can be achieved by plotting the values of both $\tilde{C}$ and C'' as a function of distance. The data obtained for the dog from test series II and III are shown in figure 10 and for the manikin in figure 11. The data for the black disk from test series II are plotted in figure 12. There was no evidence that results from either VTE procedure agreed better with values of C'' in the tests of the dog and of the black disk. However, data obtained for the manikin under the *final* VTE procedure agreed with the predicted visibility indices better than data obtained under the original VTE procedure.

The data shown by solid lines in figures 10, 11, and 12 were of considerable intrinsic interest because they represented the expected variation in visibility as a function of distance. The variations in C'' with distance are explained in the following terms: For the dog and black disk, visibility decreased slowly as a function of distance because of decreased angular size. In addition, visibility increased somewhat whenever the object was nearer than a luminaire on the same side. At this location, the objects received little illumination and, therefore, were very dark and had comparatively high negative contrast. The test in which the manikin was used produced a large sinusoidal variation in visibility as a function of distance and a superimposed general decrease in visibility as a function of distance because of the decrease in size. The locations having peak visibility corresponded to locations in which the manikin was slightly beyond a luminaire on the same side. In this location the manikin had a high degree of illumination, was very bright, and had high positive contrast to the background.

Required Illumination for Roadway Visual Tasks

On the basis of the preceding analysis, the values of $\tilde{C}$ obtained under the *final* VTE procedure seemed at least somewhat more valid than those obtained under the original procedure. Also, the two VTE procedures effected equivalent results for the shorter distances between observer and task. In

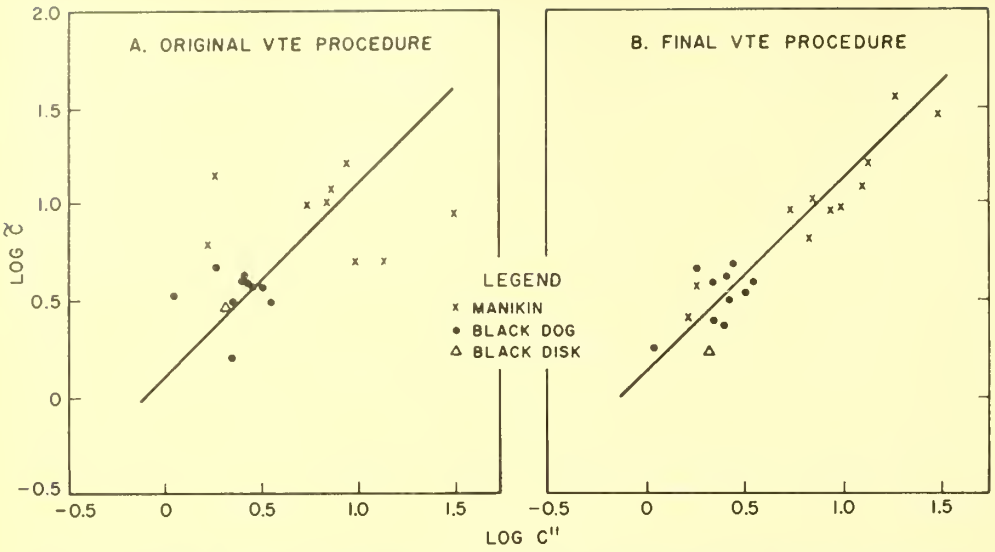

Figure 9.—Variation log $\tilde{C}$ as a function of corrected, computed equivalent contrast, log C''.

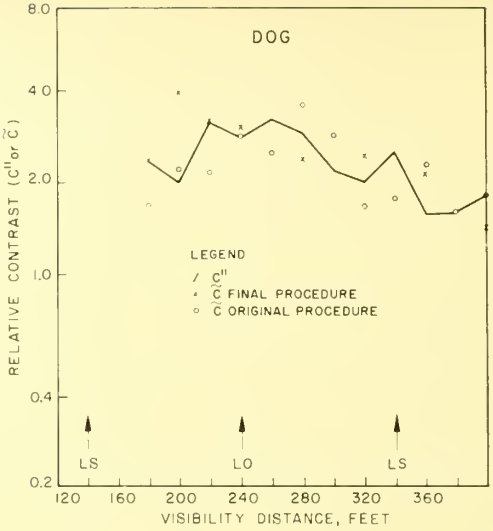

Figure 10.—Relative C'' and $\tilde{C}$ as functions of visibility distance, for a dog.

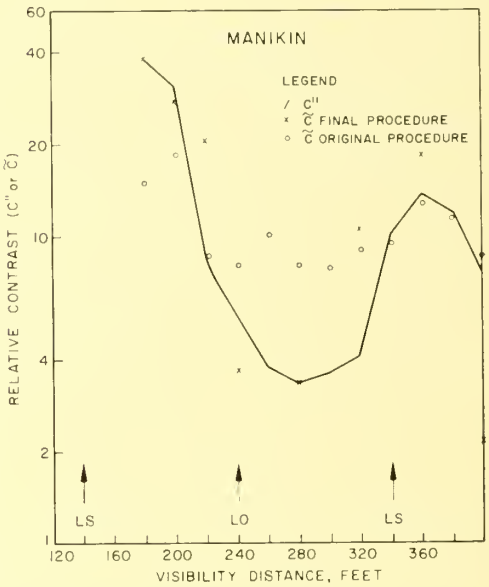

Figure 11.—Relative C'' and $\tilde{C}$ as functions of visibility distance, for a manikin.

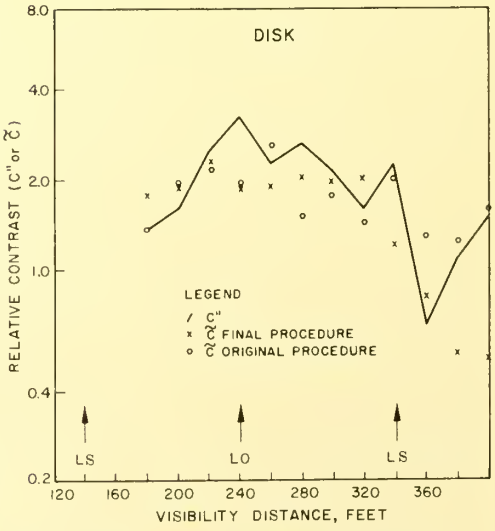

Figure 12.—Relative C'' and $\tilde{C}$ as functions of visibility distance, for a disk.

the test made with the manikin, the two VTE procedures produced approximately equivalent data for distances less than 220 feet. In the tests made with the dog and disk, the cutoff point was about 320 feet. By keeping these two findings in mind, it was then possible to sort through all the data obtained in the three series of measurements and attempt to determine what illumination (E_r and E_r') would have been required to bring the performance of these tasks to the assumed criterion level.

Test series I and II

Because it was concluded that the experiments of test series I, in which the original VTE procedure was used, distorted the visibility indices, at least for the longer distances, it was decided to restrict the use of test series I data to distances of less than 220 feet. The comparison between the original and *final* VTE procedures indicated that these two yielded equivalent results under these conditions; therefore, data at two distances—180 and 200 feet—were used.

For all experiments of test series II, the *final* VTE procedure was used, so all distances were suitable in computing values of E_r and E_r'. The different roadway conditions used during test series I were studied in test series II for a distance of 180 feet only. In addition, the dog, manikin, and black disk were studied at various distances for one illuminant-pavement combination.

Several analyses involving the data from test series I and II can be presented before discussing data from test series III because in test series III only the dog and manikin were studied under one illuminant-pavement combination. Therefore, test series I and II data contain the only information on other tasks, illuminants, and pavement. Values of E_r and E_r' for these tasks are summarized in table 1, and values for the dog and manikin obtained in the same tests are presented for comparison. The results show that different visual tasks occurring on the roadway require illumination that ranges from 0.3 to nearly 1,000 footcandles. The presence of some high values was not surprising because the more difficult roadway tasks seem at least as difficult as some of the tasks that were studied indoors and produced equally high values. From among the tasks studied, the two chosen for major emphasis—the dog and manikin—were analyzed as being a fair representation of the task of mean difficulty. All the tasks were chosen as being typical of collision obstacles.

The amount of disability glare for different roadway conditions was analyzed, and values of K' are shown in table 2. Disability glare differed significantly with the type of illuminant, being least for incandescent, a little worse for mercury, and considerably worse for fluorescent illumination. Disability glare was considerably worse on asphalt than on concrete pavements. This difference was expected because the luminaires, relative to the visual environment, seemed to be brighter when seen against the pavement material having the lower reflectance. Disability glare was also worse in the tests on the manikin than on the dog; this could have been predicted because the line of sight was elevated more for viewing the manikin than the dog.

Data on the effect of luminaire spacing is presented in table 3. To see the dog, more footcandles were required when luminaires were spaced 200 feet rather than 100 feet apart, but markedly lower illumination was required to see the manikin where the luminaires were farther apart. The differences in the effect on the illumination required to see the dog were probably not significant, but the differences required to see the manikin were. These results are explained in these terms:

A luminaire was located 40 feet in front of the object in each test. The difference in luminaire spacing, therefore, caused a difference in the distance to the first luminaire *behind* the object. The manikin was seen as an object brighter than its background because the luminance contrast was larger when the

uminaires were spaced farther apart. The manikin's luminance was unaffected by spacing but its background was darker when the luminaires were farther apart. The dog was seen as an object darker than its background because the wider spacing of luminaires reduced luminance contrast by reducing background luminance. This analysis of luminaire spacing has no generality beyond the situation tested and depends decisively upon the fact that in each test a luminaire was located 40 feet in front of the object. Had the VTE and object positions been altered, a very different result might have been obtained. This analysis demonstrated the danger of generalizing from data based on tests in only one location beneath the luminaires. Data on the task at several locations within a single cycle of the luminaires was very necessary, and the need for such data was part of the reason for conducting test series III.

Required illumination for the two major objects located in the driving and curb lanes were computed, and the results are given in table 4. In the curb lane, the object was located 5 feet to the left of the right pavement edge and was viewed from the same location in the driving lane. Had parking been allowed, this would have been the parking lane. In this test, however, no cars were parked and the lane could have been used to drive in. The values of illumination for curb and driving lanes refer particularly to the lighting needed in the respective lanes. Thus, in interpreting requirements for illumination in the curb lane, it was necessary to consider how much was produced by the lighting system in the curb lane and how much was needed.

Values of the illumination required for each object, considering disability glare, E_r', were approximately 3 times higher for the curb lane than the driving lane. Values of the required illumination, not considering glare, E_r, were approximately 2.2 times higher in the curb lane than in the driving lane. Analysis of these data, therefore, showed that a portion of the difference in requirements for illumination under the two conditions was the result of a difference in disability glare But other factors must have been at work.

The values of B_r and B_r' were higher in the curb lane when the dog was the object than in the driving lane, thus indicating that the task was more difficult in the curb lane. No consistent differences in B_r and B_r' were produced by the data about the manikin. Therefore, the visual tasks studied were at least as difficult, if not more so, in the curb lane. Also, the luminaires were less effective in producing luminance in the curb lane than in the driving lane. Together, these three factors probably account for the apparent requirement for more illumination in the curb lane for the same performance level as in the driving lane. Because more illumination was required in the curb lane, the resultant lighting problem becomes doubly difficult as in most conventional lighting systems the curb lane will have less illumination than the driving lane.

Table 1.—Illumination required to see objects 180 to 200 feet away in driving lane [1]

Object	Illumination required		Disability glare constant (K')
	No disability glare (E_r)	Disability glare (E_r')	
	Footcandles	*Footcandles*	
Auto	0.312	0.341	1.39
Manikin, light coat	.349	.358	1.13
Manikin, gray coat	.387	.414	1.35
Cone marker	.415	.436	1.21
Dog, light	1.30	1.52	1.20
Dog, black	1.64	1.80	1.18
Bicycle	7.23	10.8	1.23
Brick	782	>926	1.13

[1] Data shown are the mean values of results obtained from tests I and II, on asphalt pavement, 100-foot spacing of luminaires.

Table 2.—Disability glare constant (K') when observed objects are in driving lane [1]

Pavement and objects	Illuminant			
	Incandescent		Mercury	Fluorescent
	6,000 lumen	15,000 lumen		
Asphalt:	K'	K'	K'	K'
Dog	1.18	1.18	1.12	1.78
Manikin	1.23	1.35	1.38	2.00
Concrete:				
Dog	1.02	1.05	1.15	1.38
Manikin	1.12	1.12	1.35	1.48
Mean [2]	1.16		1.25	1.66

[1] Data from test series I, 100-foot spacing of luminaires.
[2] Means for the disability glare constant for type of pavement and objects are: Pavement—asphalt, 1.40; concrete, 1.21; object—dog, 1.23; manikin, 1.38.

Table 3.—Illumination required to see object when luminaires are at two different spacings [1]

Test series	Visibility distance	Illumination required			
		100 feet between luminaires		200 feet between luminaires	
		No disability glare (E_r)	Disability glare (E_r')	No disability glare (E_r)	Disability glare (E_r')
DOG					
	Feet	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*
I	180	0.649	0.664	1.30	1.42
I	200	.649	.664	1.37	1.50
II	180	1.27	1.33	.991	1.06
II	200	.986	1.03	.589	.646
Mean		0.889	0.922	1.06	1.16
MANIKIN					
I	180	0.443	0.463	0.135	0.144
I	200	.461	.471	.283	.311
II	180	1.17	1.32	.235	.240
II	200	.601	.672	.379	.417
Mean		0.669	0.732	0.258	0.278

[1] Source of light, 15,000-lumen incandescent illuminants on concrete pavement.

The illumination required by use of the different illuminants studied is given in table 5. Analysis of the E_r values shows that the illuminants may differ in complex ways even without the disability glare being a factor. Fluorescent illuminants seemed to be superior to incandescent, and mercury illuminants were inferior to incandescent illuminants for objects such as the manikin. Because both mercury and fluorescent illuminants produce more disability glare than incandescent, a study of the values of E_r' shows that for both the mercury and fluorescent illuminants more illumination was required than for the incandescent. The data were somewhat erratic and the differences should be applied with considerable caution, especially as only one type of fixture for each illuminant was compared in this study.

The results of an analysis of the illumination required on asphalt and concrete pavement surfaces are given in table 6. A study of the values of both E_r and E_r' shows that less

Table 4.—Illumination required to see objects in curb lane and driving lane: Source of light, 15,000-lumen incandescent illuminants

Pavement type and test series	Visibility distance	Illumination required				Disability glare constant (K′)	
		No disability glare (E_r)		Disability glare (E_r')			
		Driving lane	Curb lane	Driving lane	Curb lane	Driving lane	Curb lane
DOG							
Asphalt:	*Feet*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*		
I	180	1.21	2.00	1.32	2.58	1.18	1.35
I	200	1.05	1.81	1.10	2.28		
II	180	2.65	11.1	2.98	17.6		
Concrete:							
I	180	.649	.721	.664	.810	1.05	1.13
I	200	.649	.810	.664	.930		
II	180	1.27	.939	1.33	.985		
Mean		1.25	2.89	1.34	4.20	1.12	1.24
MANIKIN							
Asphalt:							
I	180	0.360	1.32	0.395	1.75	1.35	1.59
I	200	.349	2.83	.374	4.29		
II	180	.451	.455	.472	.498		
Concrete:							
II	180	1.17	.381	1.32	.399	1.12	1.20
Mean		0.582	1.25	0.640	1.74	1.24	1.40

Table 5.—Illumination required to see objects in driving lane under different illuminants

Pavement type and test series	Visibility distance	Illumination required							
		No disability glare (E_r)				Disability glare (E_r')			
		Incandescent		Mercury	Fluorescent	Incandescent		Mercury	Fluorescent
		6,000 lumens	15,000 lumens			6,000 lumens	15,000 lumens		
DOG									
Asphalt:	*Feet*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*
I	180	1.84	1.21	1.53	0.876	2.16	1.32	1.83	1.40
I	200	.984	1.05	.763	.739	1.10	1.10	.875	1.02
II	180	.982	2.65	2.30	.754	1.10	2.98	2.96	1.17
Concrete:									
I	180	.762	.649	.616	.917	.780	.664	.692	1.21
I	200	.558	.649	.483	1.08	.558	.664	.517	1.46
II	180	.476	1.27	.580	.685	.481	1.33	.636	.873
Mean		1.09		1.05	0.842	1.18		1.25	1.19
MANIKIN									
Asphalt:									
I	180	0.213	0.360	0.403	0.318	0.218	0.395	0.432	0.429
I	200	.443	.349	1.26	.636	.474	.374	1.48	.920
II	180	.460	.451	.603	.435	.482	.472	.654	.601
Concrete:									
I	180	.311	.443	.708	.295	.318	.463	.852	.339
I	200	.268	.461	.444	.562	.274	.471	.498	.677
II	180	.540	1.17	1.18	.423	.598	1.32	1.31	.509
Mean		0.456		0.766	0.445	0.488		0.871	0.579

Table 6.—Mean values of illumination required to see objects on different types of pavement—based on data in table 5

Object	Illumination required			
	No disability glare (E_r)		Disability glare (E_r')	
	Asphalt	Concrete	Asphalt	Concrete
	Footcandles	*Footcandles*	*Footcandles*	*Footcandles*
Dog	1.31	0.727	1.58	0.822
Manikin	.494	.567	.577	.636

illumination was required on concrete than on asphalt pavement when the dog was the object, but when the manikin was the object more illumination was required on the concrete pavement. This finding is explained by the relative reflectances of the objects and the pavement surfaces. The dog was dark and matched the asphalt considerably better than the concrete in reflectance. Therefore, the dog was more difficult to see on asphalt and considerably more illumination was required. Because the manikin was comparatively light and matched concrete somewhat better than asphalt, the manikin was somewhat more difficult to see on concrete and somewhat more illumination was required. This analysis explains clearly that the illumination required on the two different pavements depends intrinsically upon the object on the roadway, and that no general statement comparing the two types of pavement can be made accurately.

Test series III

Values of E_r and E_r' for the measurements of test series III are given in tables 7 and 8. These values represent all the data from the third series of tests; the *final* VTE procedure was used exclusively. Illumination required is presented for each object and each distance; these data represent averages for the 11 different locations of the VTE as related to the luminaires. The values given, however, are restricted to the tests of incandescent luminaires and asphalt pavement. The same conclusion—that the manikin was somewhat less difficult to see on the asphalt pavement than the dog—can be drawn from the test series III data for distances of less than 300 feet. At longer distances, however, the manikin was no longer seen against the pavement in most tests. The small, white, frame house in the woods at the far end of the roadway may have been a critical factor.

Expressing data on roadway requirements for lighting in terms of pavement luminances rather than in illumination units, as has been done in the study reported here, is of considerable contemporary interest. Although the eye is concerned with luminances and not illumination requirements, the data herein are not presented in terms of luminances because:

First, although it is possible to design a lighting installation in terms of the illumination, it is difficult, if not impossible, to design it to provide specified luminances because of the lack of complete knowledge of the reflectance characteristics of pavement surfaces. Second, use of luminances could influence illuminating engineers so that they might forget that illumination has two functions in roadway lighting: (1) To produce pavement luminance; and (2) to produce object contrast. An analysis of the data compiled for this article suggests that contrast is more important than luminance.

The values of E_r and E_r' given in tables 7 and 8, the authors believe, are the best evaluations of the illumination needed to see the dog and manikin under incandescent luminaires and on asphalt pavement. These

data can be used to provide a basis for establishing suitable illumination levels for roadway lighting. Consider first the test at a 200-foot distance—the data from test series I and II can be used with confidence for this distance or shorter distances. The data from test series III applied only to objects seen on the asphalt pavement under incandescent illumination. The data from test series I and II were used to define ratios relating illumination requirements for the other illuminant-pavement combinations to the incandescent illuminant-asphalt pavement condition. These ratios were then used to adjust the data from test series III to apply to other illuminants and/or pavements. The factors are summarized in table 9 for useful combinations of illuminant and pavement. The factors for the dog and manikin are maintained separately for E_r and E_r' values, respectively. For test series III, the average values of E_r and E_r' for the incandescent-asphalt combination were taken from tables 7 and 8.

Using the factors, estimated values of E_r and E_r' were computed for each combination of illuminant and reflectance and are given in table 9. The values given in this table represent the estimates of the illumination required for targets located in the driving lane, and each combination of illuminant and reflectance is given equal weight. The E_r' values of 1.30 footcandles required to see the dog and 1.85 footcandles required to see the manikin represent the summary result of the entire study of roadway visual tasks. Of course, as was pointed out, all that can be suggested is to specify the illumination required for adequate visual performance for a *particular* illumination geometry and location of object and observer. Thus, the illumination units have no generality and cannot be used except in terms of similar conditions of illumination and viewing. Where geometry is different, as at other sites tested (*5*), the illumination required also was different, and average illumination could *not* be used as a reliable indicator of visibility.

An adequate understanding of the extent to which objects may be seen anywhere on the roadway when they appear without warning cannot be obtained only from the average values of illumination. To obtain some estimate of this aspect of the roadway lighting problem, values of E_r and E_r', for each of the 11 locations of the VTE used in test series III were computed for each combination of illuminant and pavement surface. The factors presented in table 9 were used to compute these data from the values of E_r and E_r' given for individual locations in tables 7 and 8. These calculations produced 66 values of E_r and E_r' for each target. They were used to generate the cumulative frequency graphs in figure 13. The ordinate in this figure represents the percentage of locations along the roadway in which the target in question was predicted to be adequately visible at 200 feet. Values of average illumination provided by the hypothetical lighting system of the same geometry are shown on the abscissa. Values of E_r' are of primary interest; however, the values of E_r are given only to show how much less illumination could be used if disability glare could be entirely eliminated from roadway lighting.

It may be of interest to evaluate the extent to which these average required illuminations depend on the distance at which objects must be seen. The average values of E_r and E_r' from tables 7 and 8 may be expressed as a ratio of the average value for the 200-foot distance. Such ratios are plotted in figures 14 and 15. It is clear that the illumination required differs only a little between 180 and

Table 7.—Illumination required for observer at different locations to see objects at various distances, no disability glare, test series III

Observer location	Illumination required							
	Visibility distance, feet—							
	180	200	220	240	280	320	360	400
Dog								
	Footcandles	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*	*Footcandles*
1	1.33	0.552	0.965	1.16	2.30	1.86	1.66	2.71
2	1.03	1.32	1.04	1.27	3.16	1.92	2.34	2.85
3	1.60	1.02	2.51	1.92	1.82	1.78	2.20	3.01
4	1.05	1.45	1.42	1.96	2.24	1.51	2.13	1.88
5	1.27	.890	1.38	1.18	1.25	1.15	1.34	1.44
6	.815	.774	1.60	.980	1.14	2.08	1.77	1.90
7	1.09	1.34	1.06	.975	1.48	1.65	2.50	3.37
8	.772	1.10	1.30	2.00	2.75	2.38	4.07	4.55
9	1.58	1.62	3.02	3.90	3.25	3.03	4.11	1.14
10	1.59	1.42	3.14	2.35	2.76	2.70	2.88	4.29
11	1.76	2.22	2.18	2.00	3.16	3.89	5.19	5.21
Mean	1.26	1.25	1.78	1.79	2.30	2.18	2.74	2.94
Manikin								
1	0.312	0.600	0.791	1.23	1.56	1.51	0.773	3.08
2	.834	1.05	1.23	1.41	1.20	1.42	.956	3.58
3	.842	.840	1.23	3.12	2.60	5.66	2.32	2.27
4	.806	1.06	2.32	1.28	1.08	.724	2.68	2.58
5	.696	1.61	1.12	1.51	.495	1.58	3.20	12.2
6	1.06	1.40	1.83	.914	1.12	3.00	3.00	2.66
7	.825	1.22	1.26	.449	1.45	2.89	14.8	24.8
8	1.96	1.05	.775	1.17	2.56	8.03	7.08	3.88
9	1.02	.721	1.20	1.17	2.30	8.90	34.6	2.44
10	.845	1.57	2.16	2.20	2.20	24.6	10.2	34.6
11	.510	.893	2.26	2.10	2.11	1.92	1.33	2.20
Mean	0.882	1.09	1.47	1.50	1.70	5.48	7.35	8.57

Table 8.—Illumination required for observer at different locations to see objects at various distances, disability glare (K'), test series III

Observer location	Illumination required								
	K'	Visibility distance, feet—							
		180	200	220	240	280	320	360	400
Dog									
		Foot-candles	*Foot-candles*	*Foot-candles*	*Foot-candles*	*Foot-candles*	*Foot-candles*	*Foot-candles*	*Foot-candles*
1	1.20	1.56	0.591	1.11	1.40	2.84	2.24	1.99	3.56
2	1.29	1.30	1.75	1.34	1.60	4.27	2.42	3.02	3.76
3	1.41	2.21	1.32	3.79	2.90	2.51	2.40	3.56	5.11
4	1.66	1.42	2.30	2.25	3.41	4.17	2.24	3.78	3.50
5	1.59	1.97	1.23	2.09	1.82	1.77	1.73	2.06	2.18
6	1.62	1.15	1.09	2.80	1.26	1.68	3.70	3.00	3.22
7	1.62	1.54	2.03	1.37	2.32	2.46	2.74	4.76	4.15
8	1.32	.930	1.26	1.56	2.63	3.88	3.36	6.60	4.98
9	1.32	2.03	2.04	4.79	7.61	5.15	4.80	4.39	2.93
10	1.29	1.96	1.75	4.59	3.40	4.09	3.91	3.09	4.60
11	1.32	2.22	2.99	3.09	2.82	4.68	4.79	6.25	7.89
Mean	------	1.66	1.67	2.62	2.83	3.41	3.12	3.86	4.17
Manikin									
1	1.20	0.312	0.600	0.800	1.44	1.88	1.54	0.781	3.96
2	1.29	.841	1.26	1.52	1.78	1.52	1.45	.969	4.95
3	1.41	.853	.840	1.26	4.73	3.84	5.66	3.52	3.35
4	1.66	.845	1.12	4.64	1.51	1.19	.758	6.13	5.60
5	1.59	.730	2.44	1.57	2.39	.506	1.80	6.38	52.3
6	1.62	1.57	2.22	3.18	1.02	1.20	3.28	4.15	4.21
7	1.62	.883	1.77	1.38	.449	2.24	5.38	64.4	86.0
8	1.32	2.58	1.08	.791	1.22	3.62	13.3	13.5	5.25
9	1.32	1.04	.729	1.25	1.22	3.11	13.4	104.	3.08
10	1.29	.885	1.73	2.92	2.83	2.83	57.4	16.2	45.6
11	1.32	.510	.904	3.20	2.77	2.16	2.01	1.36	2.30
Mean	------	1.000	1.33	2.05	1.94	2.19	9.64	20.2	19.7

Table 9.—*Illumination required to see each of two objects 200 feet away on different types of pavement under different illuminants*

Pavement and illuminant	No disability glare (E_r)		Disability glare (E_r')	
	Multiplication factor [1]	Illumination required	Multiplication factor [1]	Illumination required
DOG				
Asphalt:		*Footcandles*		*Footcandles*
Incandescent	1.00	1.25	1.00	1.67
Mercury	.964	1.20	1.06	1.77
Fluorescent	.772	.965	1.01	1.69
Concrete:				
Incandescent	.556	.696	.519	.867
Mercury	.536	.670	.548	.915
Fluorescent	.429	.535	.523	.873
Mean	NA	0.886	NA	1.30
MANIKIN				
Asphalt:				
Incandescent	1.00	1.09	1.00	1.33
Mercury	1.68	1.83	1.78	2.37
Fluorescent	.977	1.07	1.18	1.57
Concrete:				
Incandescent	1.15	1.25	1.10	1.46
Mercury	1.93	2.11	1.96	2.61
Fluorescent	1.12	1.22	1.30	1.73
Mean	NA	1.43	NA	1.85

[1] Ratio of illumination required for condition to that required for an incandescent source on asphalt pavement, determine from test series I and II.

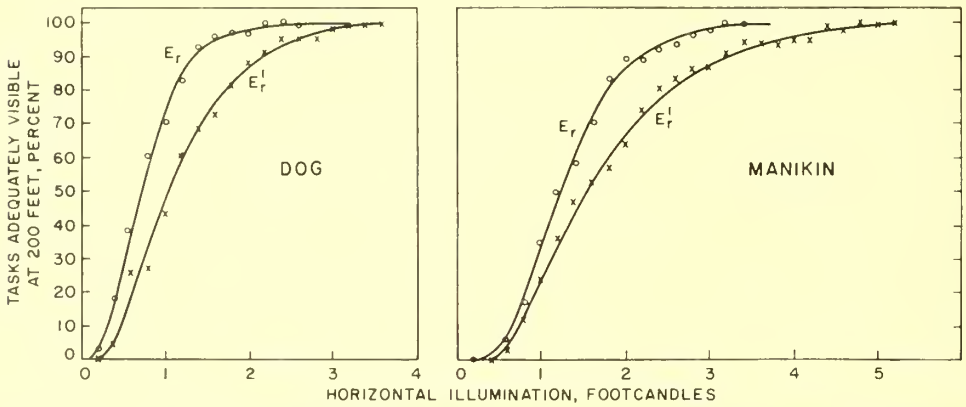

Figure 13.—Percentage of tasks adequately visible at 200 feet for different levels of horizontal illumination.

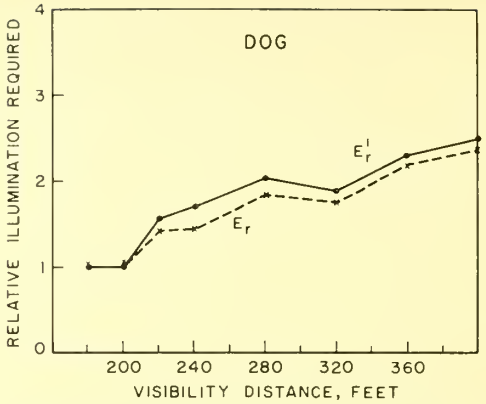

Figure 11.—Relative illumination levels required to see a dog at distances other than 200 feet.

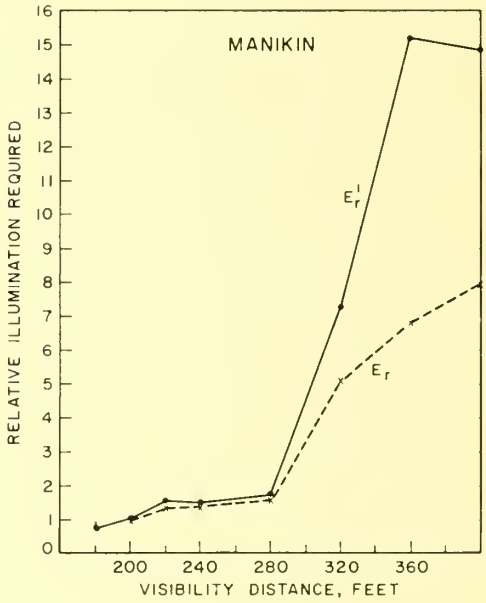

Figure 15.—Relative illumination levels required to see a manikin at distances other than 200 feet.

200 feet, but that considerably more illumination is required at distances of 300 to 400 feet than at 200 feet. To see the manikin, the increase in illumination was considerably more than the increase needed to see the dog.

In evaluating the illumination requirement data from the Hendersonville test site, generally lower illumination values were necessary to meet the same performance criterion than illumination requirements for actual highway sites in Ohio (*5*). Thus, the final required illumination values for adequate visibility reported herein are probably conservative.

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The Mendenhall Glacier and the Forest highway leading to it, Alaska.

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IN THIS ISSUE

Effect of Soil Blankets and Preloading on Settlement, *by Edward S. Barber* 249

Mechanisms of Soil-Lime Stabilization. An Interpretive Review, *by Sidney Diamon and Earl B. Kinder* 260

Experimental Isolation of Drivers' Visual Input, *by Donald A. Gordon* 266

Travel Habits in Cities of 100,000 or More, *by Mrs. Thurley A. Bostick and Thomas R. Todd* . . 274

New Publications 276

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The title sheet for vol. 33, Apr. 1964–Feb. 1966, of *PUBLIC ROADS, A Journal of Highway Research* is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by a request to the editor of the magazine, Bureau of Public Roads, Washington, D.C., 20235.

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Effect of Soil Blankets and Preloading on Settlement

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

By EDWARD S. BARBER, Highway Research Engineer,[1] Materials Division

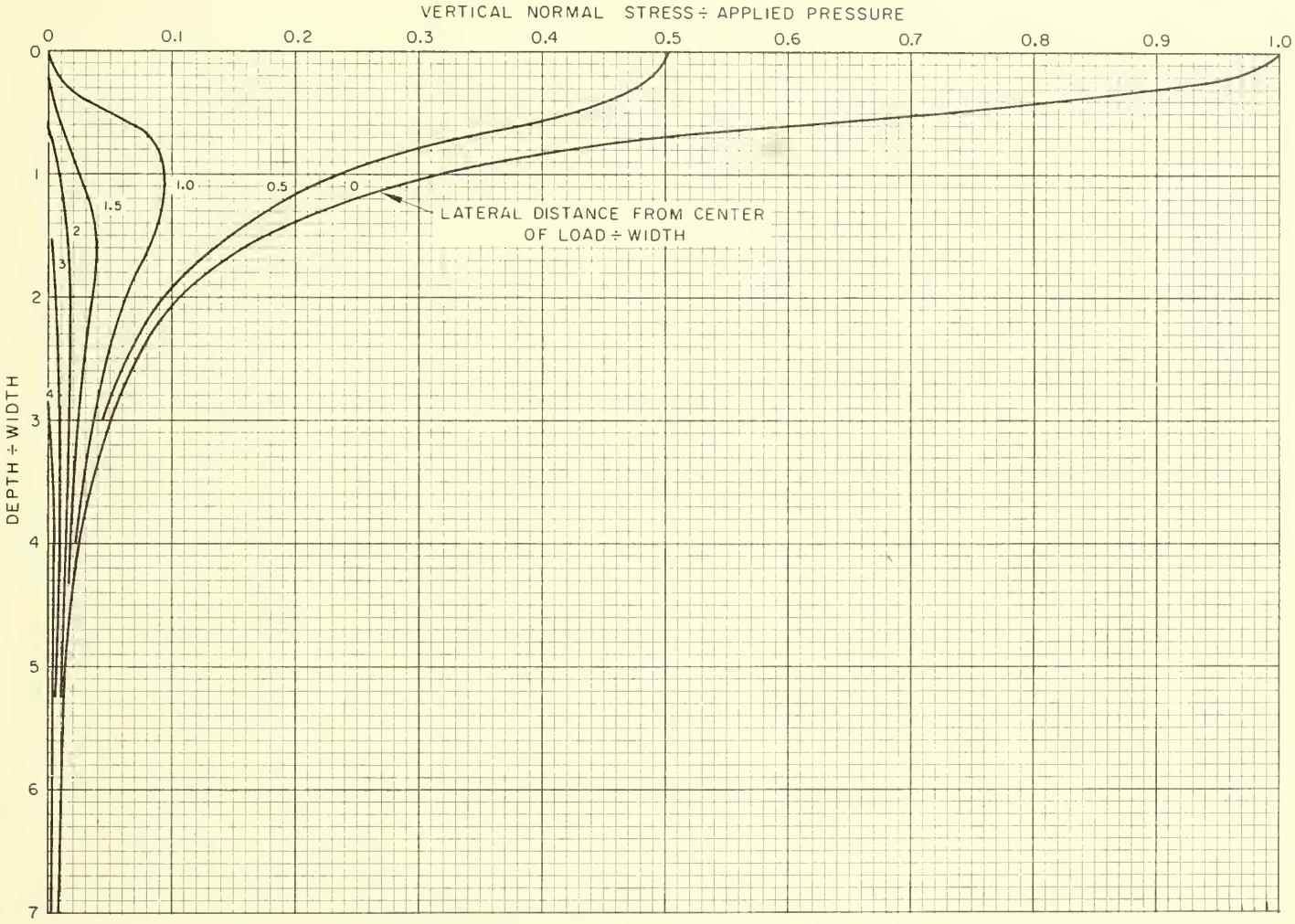


Figure 1.—Vertical normal stress from uniformly loaded square.

Highways must often be located in areas where soil is compressive and, unless some action is taken to preclude it, excessive settlement may occur. Soil blankets and natural or manmade preloads applied to compressible material often will permit placement of foundation structures on compressible material thereby effecting a saving by obviating the need for deep foundations.

This article contains charts and tables to be used in calculating the vertical stresses and settlement that occur under preloads of different shapes. The usefulness of soil blankets and preloads is discussed. A more accurate procedure for designing the duration of preload, rather than basing this duration on average consolidation, also is presented.

Introduction

FOUNDATION structures can be placed on compressible soil materials without excessive settlement if the applied stresses do not significantly exceed the stresses previously imposed on the material. A blanket of relatively incompressible soil can be used over compressible material to spread the foundation load and thereby reduce the imposed stresses, and a temporary overload on the compressible material can be used to consolidate or prestress it. A considerable cost saving has been made by the use of soil blankets and natural or manmade preloads over the cost of use of deep foundations extending below the compressible materials.

This article includes original charts and tables, based on established theory, for calculating vertical stresses and settlement under loads of different shapes. Information is given on the usefulness of soil blankets and preloads and on the error inherent in the practice of basing preload duration on average consolidation. Also, a more accurate procedure for designing preload duration is described.

Conclusion

On the basis of information presented in this article, it is concluded that, when accompanied by adequate exploration and testing, the use of soil blankets and preloads on

[1] Mr. Barber is also Associate Professor of Civil Engineering at the University of Maryland, College Park, Md.

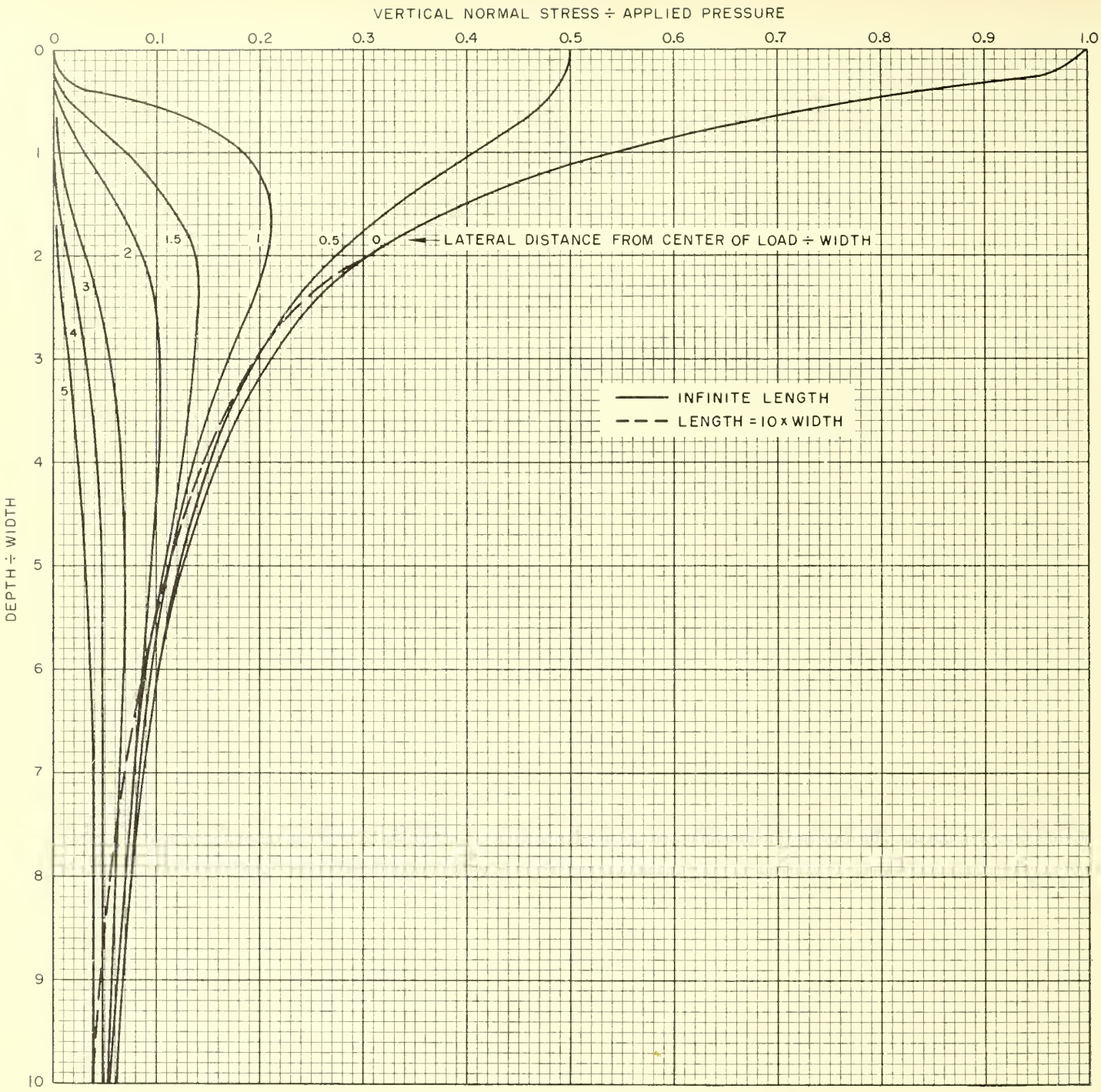


Figure 2.—Vertical normal stress from uniformly loaded rectangle.

compressible soil can sometimes provide a satisfactory foundation. Use of the procedures described herein will often obviate the need for construction of deep foundations below compressible material.

Vertical Stresses

In soils not close to failure, stress distribution is generally calculated on the basis of the theory of elasticity (*1*),[2] which has been shown by test to be applicable to soil (*2*). Based on integration of the Boussinesq formula, the vertical stress produced by a uniform pressure over a square area in a homogeneous elastic medium is shown in figure 1. Also shown is the maximum stress under the center of the load and at different lateral distances. Similar stresses are shown in figure 2 for a uniform strip load of infinite length, and the maximum stress also is shown under a uniformly loaded rectangle whose length is 10 times the width. As depth increases, the total load becomes more important than the applied unit stress; thus, to produce a given stress at given depth, as the loaded area is decreased a higher unit pressure must be applied. For example, consider an inplace, loose, granular blanket 10 feet thick in which it is desired to support strip footings 2 feet wide, 2.5 feet below the surface and loaded to 2 kips per square foot. At the bottom of the blanket, the depth divided by the width is:

$$(10-2.5)\div 2=3.75$$

From data in figure 2, which shows a maximum ratio of vertical normal stress divided by the applied pressure of 0.17 (under the center of the footing, curve marked 0), a vertical stress of 0.17×2 equals 0.34 kip per square foot (k.s.f.) at the effective depth of 7.5 feet, can be determined.

To determine the rolling pressure required at the surface of the blanket to produce essentially the same stress at the bottom of the blanket, use data in figure 1. Here the effective depth is 10 feet or the same as the

[2] Italic numbers in parentheses indicate the references listed on p. 258.

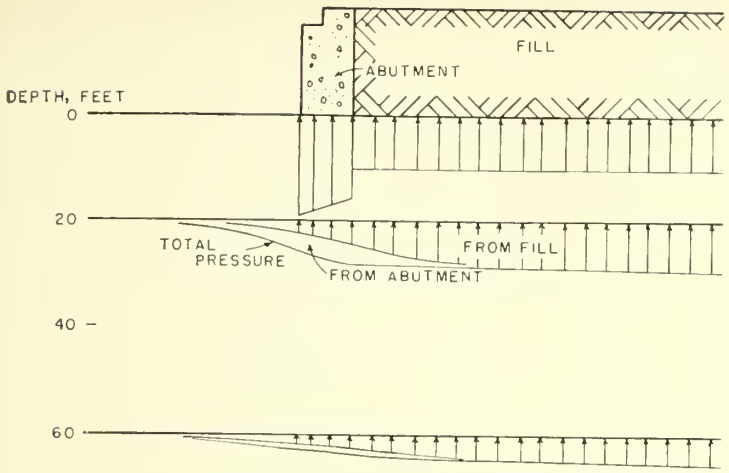


Figure 3.—Vertical pressure at different depths in soil from 10-by-30-foot abutment and a fill 20 feet high and an average of 60 feet wide.

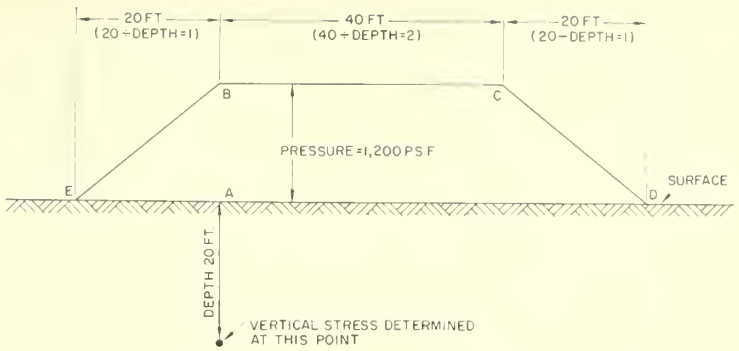


Figure 5.—Cross section of strip load.

thickness of the blanket. Assume that a 40-ton roller acts on a 4-foot-square area with a pressure of 5 k.s.f. It can be determined by use of data from figure 1 that a pressure at a 10-foot depth of 0.07(5) equals 0.35 k.s.f. Thus, 5 k.s.f. on a small area is required to obtain the same pressure 10 feet below the surface as 2 k.s.f. on a long strip at an effective depth of 7.5 feet. Figure 3 illustrates how the fill behind an abutment is the major contributor of stress at any but shallow depths. For a grade separation, temporary fill across the gap between two approach fills may be used to preload a deep compressible layer to stresses equal to those produced by bridge loads.

For determining the preload required for strip loads of nonuniform cross section, the influence chart, figure 4, was developed. To determine the stress at a given distance below point **A** (lower left corner), the load cross section is drawn on the chart using lateral distance from **A** divided by depth as the horizontal scale and any convenient vertical scale for pressure. The stress factor is the sum of influence blocks covered by the cross section, multiplied by the appropriate influence factors. The vertical stress at the specified depth is then the product of the pressure, corresponding to unit pressure on the chart, and the stress factor.

For example, consider the long strip load shown in figure 5. For stress at a depth of 20 feet the lateral distances are divided by 20 feet to obtain lateral distance ratios of 1 from **E** to **B**, 2 from **B** to **C**, and 1 from **C** to **D**. Using these ratios and arbitrarily plotting the maximum pressure at half the height of the chart in figure 4, points **B**, **C**, and **D** are located with respect to **A**. **E** would be to the left of **A**, but since the chart, if extended, would be symmetrical about **A**, **E** is plotted to the right as though the loading diagram were folded about **AB**. The number of influence blocks used to determine a total stress factor of 0.369 was:

Block			Stress factor
Value	*Number* **ABCD**	*Number* **ABE**	*Total*	
0.001	150	112	262	0.262
0.0005	188	26	214	0.107

Fractional values of blocks were used in balancing deficiencies in the 0.0005 blocks. The pressure corresponding to unit pressure is

$$(1\div 0.5)\,1{,}200 = 2{,}400 \text{ p.s.f.}$$

The vertical stress 20 feet below point **A** shown in figure 5 then would be

$$2{,}400\times 0.369 = 886 \text{ p.s.f.}$$

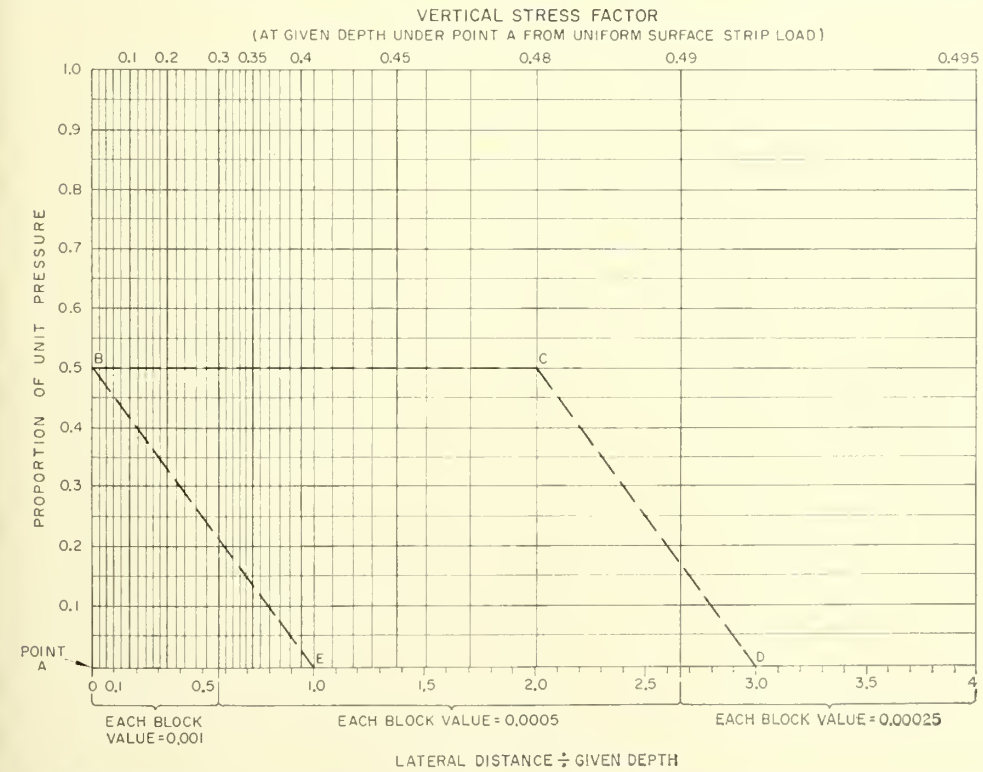

VERTICAL STRESS FACTOR
(AT GIVEN DEPTH UNDER POINT A FROM UNIFORM SURFACE STRIP LOAD)
0.1 0.2 0.3 0.35 0.4 0.45 0.48 0.49 0.495
1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1
PROPORTION OF UNIT PRESSURE
B
C
E
D
POINT A
0 0.1 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4
EACH BLOCK VALUE=0.001
EACH BLOCK VALUE=0.0005
EACH BLOCK VALUE=0.00025
LATERAL DISTANCE ÷ GIVEN DEPTH

Figure 4.—Chart for computing vertical stress at any point from surface strip load, stress = pressure × factor.

Settlement

For consolidation and no lateral displacement, Poisson's ratio equal to zero is used. For this condition, settlement is proportional to the area between the vertical stress curve and the axis of depth, as determined from use of information in figures 1 or 2. Settlement factors based on integration of the areas between stress curves and depth axes are shown in figure 6 for different rectangles loaded uniformly. The effect of a moment is also shown in figure 6. For example, consider a rectangle uniformly loaded to 4 kips per square foot (inset in fig. 6) where **B** = 20 feet and **L** = 40 feet. The settlement is to be determined under the point **A** that is caused by compression of a layer between 10 and 20 feet deep (points **C** and **A**) for which the strain per unit pressure, m_v, is 1 percent per kip per square foot. In figure 6, for

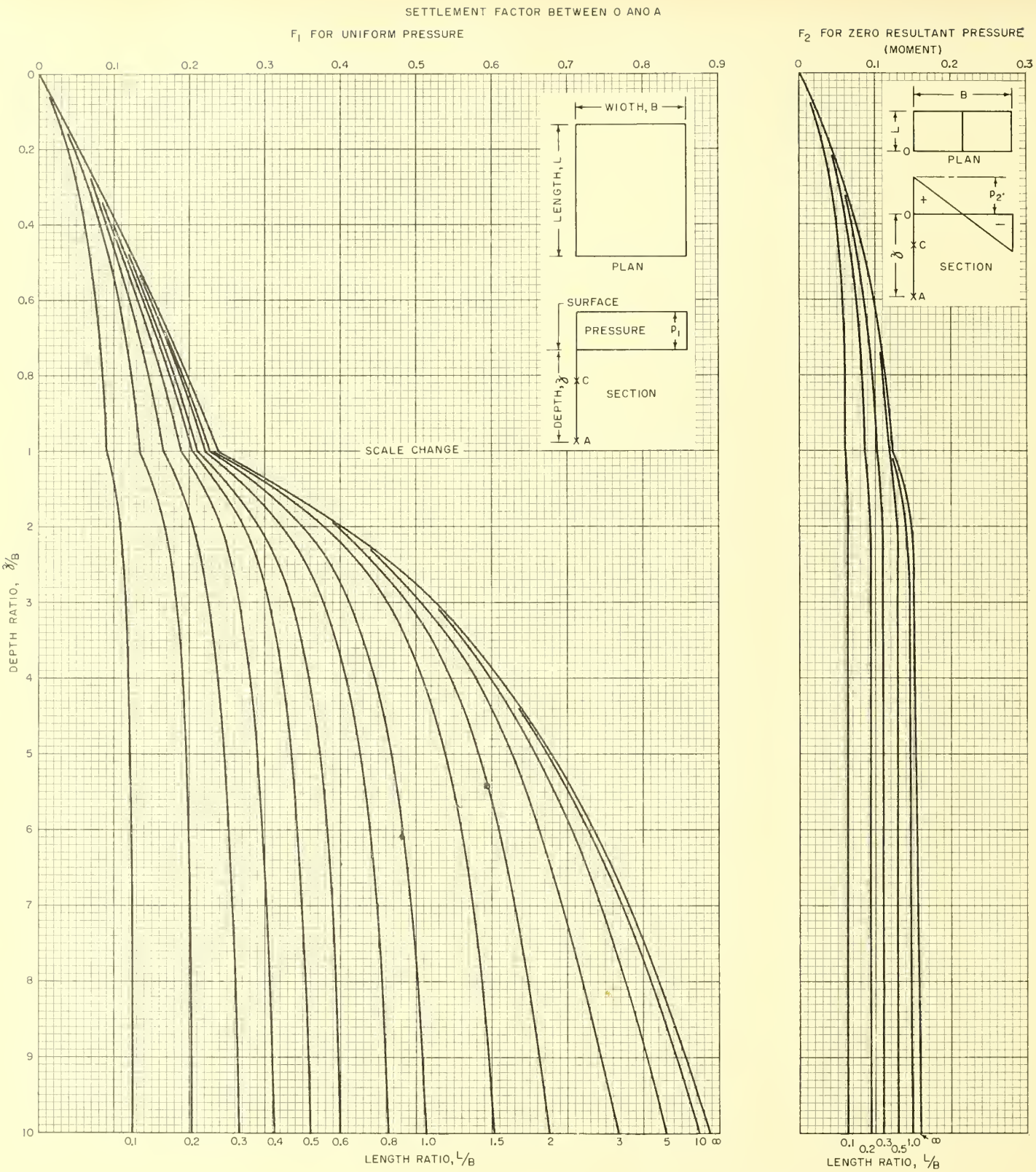
$$\mathbf{L}\div\mathbf{B} = 40\div 20 = 2,$$

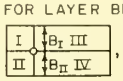
read $\mathbf{F}_1$ values of 0.13 and 0.24 respectively for

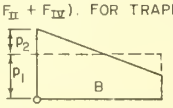
$$y\div\mathbf{B} = 10\div 20 = 0.5 \text{ and } 20\div 20 = 1.$$

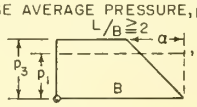
Using the equation at the bottom of figure 6, the settlement is

$$0.01\times 4\times 20\times(0.24-0.13) = 0.088 \text{ feet.}$$



SETTLEMENT, $S = m_v p B F$. m_v = STRAIN PER UNIT PRESSURE INCREASE FROM CONSOLIDATION TEST CORRESPONDING TO AVERAGE PRESSURE, $\frac{pF}{\vartheta/B}$. VERTICAL PRESSURE AT A POINT IS p X SLOPE OF CURVE = $p\frac{dF}{d(\vartheta/B)}$. FOR LAYER BETWEEN C AND A, USE $F = F_A - F_C$. FOR A POINT NOT BELOW CORNER, USE COMBINED AREAS THUS: FOR 

, USE $BF = B_I(F_I + F_{III}) + B_{II}(F_{II} + F_{IV})$. FOR TRAPEZOIDAL PRESSURE DISTRIBUTION, USE AVERAGE PRESSURE, p, AND COMBINE F_1 AND F_2, THUS: FOR 

USE $F = F_1 + \frac{p_2}{p_1} F_2$ AND $p = p_1$, FOR 

, $F = F_1 + \frac{a}{B} F_2$ AND $p = p_1 = p_3\left(1 - \frac{a}{2B}\right)$.

Figure 6.—Graph of settlement under corner of load on rectangular area, Poisson's ratio = 0.

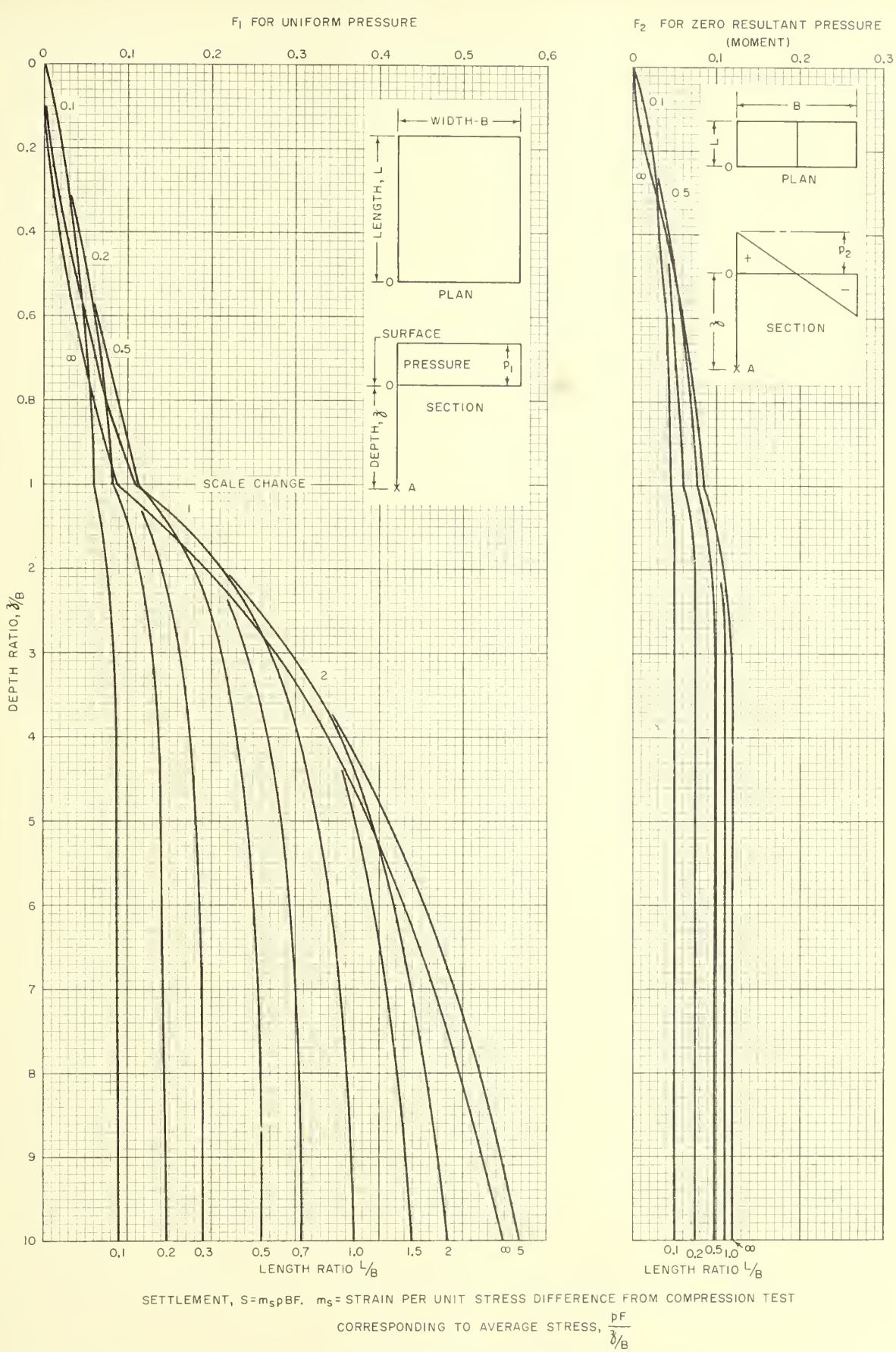


Figure 7.—Graph of settlement under corner of load on rectangular area, Poisson's ratio=0.5.

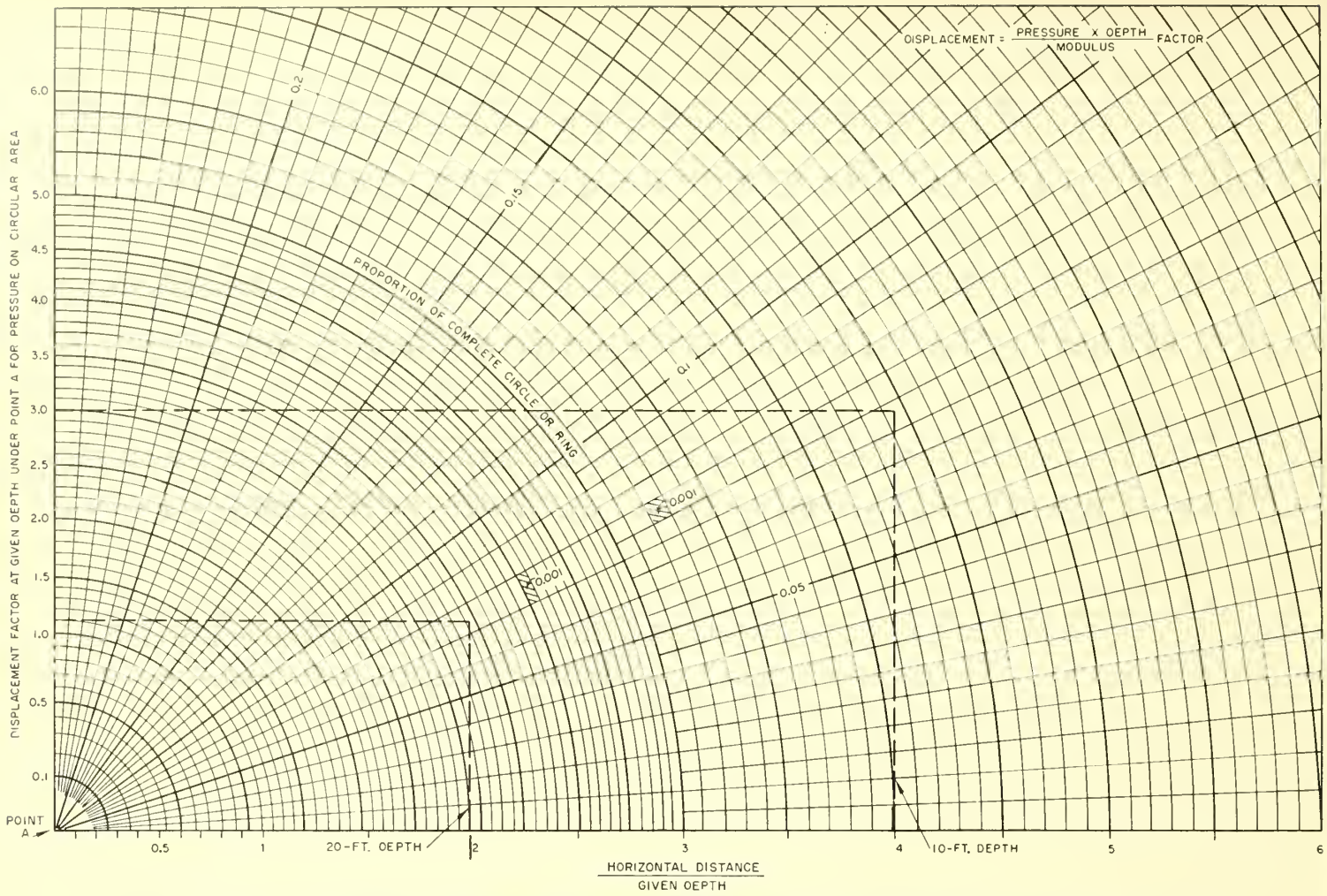


Figure 8.—Chart for computing vertical displacement below the surface from finite surface load, Poisson's ratio=0.

For Poisson's ratio equals 0.5, corresponding to shear at constant volume, settlement factors are a function of both vertical and lateral normal stresses. Integration for this condition produced data for figure 7, which shows settlement factors under different loaded rectangular areas for Poisson's ratio equals 0.5.

Settlement factors for other shapes may be obtained from influence charts such as figure 8, which is similar to Newmark's chart for vertical stress (*3*). For a displacement factor for material below a given depth under point **A** (lower left corner), the loaded area is located to scale and the number of blocks covered by the area is multiplied by 0.001 (influence value per block) to obtain the factor. The displacement is calculated by this factor multiplied by applied pressure multiplied by the depth that is then divided by modulus of elasticity between the surface and the given depth.

$$\text{Displacement} = p \times j \times \mathbf{F} \div \text{modulus}.$$

Although, figure 8 information can be used for calculating settlement factor for more irregular areas; its use is illustrated by solving the problem used to illustrate figure 6.

The rectangular loaded area is plotted in figure 8 to the appropriate scale for each depth. To prevent error that possibly might be caused by nonuniform shrinkage of the paper on which figure 8 is printed, both the width÷depth and the length÷depth ratios are plotted on the bottom scale and the width÷depth ratios transferred to the vertical scale by following the appropriate curves, which were originally ares. The number of blocks covered are 1,270 and 525, respectively, for the rectangles for 10-foot depth and the

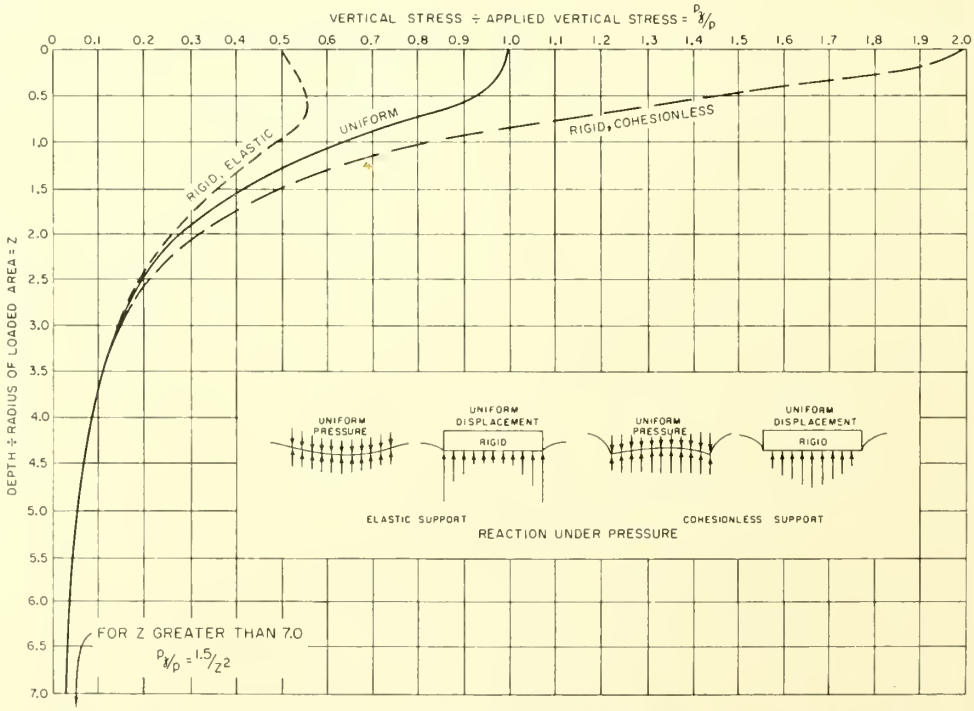


Figure 9.—Vertical stresses on axis below vertical stress over circular area at surface.

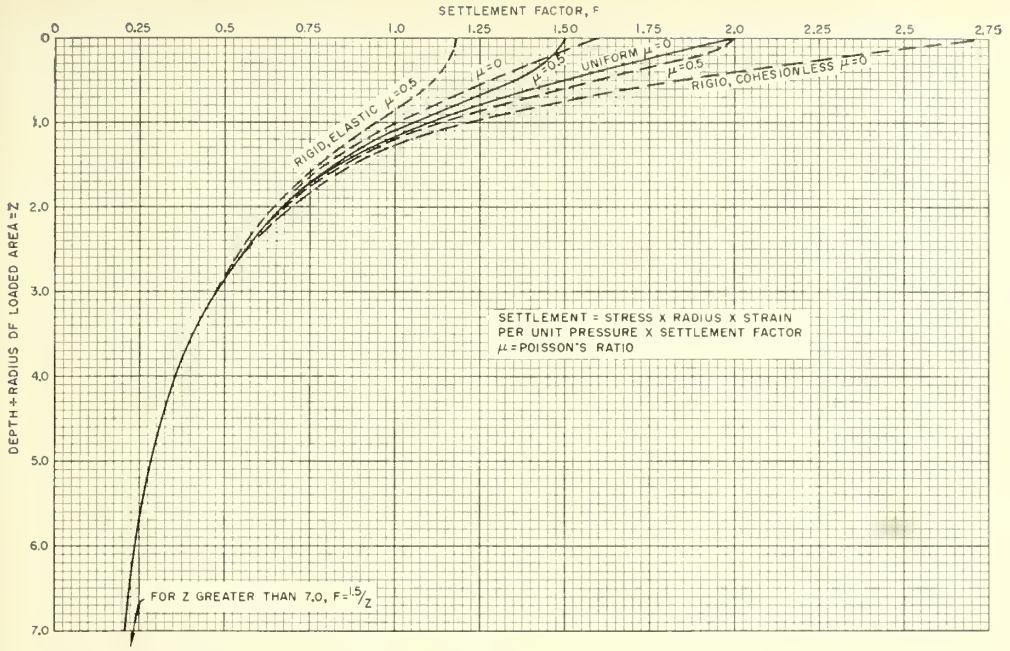

SETTLEMENT FACTOR, F
0 0.25 0.5 0.75 1.0 1.25 1.50 1.75 2.0 2.25 2.5 2.75
DEPTH ÷ RADIUS OF LOADED AREA = Z
0 1.0 2.0 3.0 4.0 5.0 6.0 7.0
RIGID, ELASTIC μ=0.5
μ=0
μ=0.5
UNIFORM μ=0
μ=0.5
RIGID, COHESIONLESS μ=0
SETTLEMENT = STRESS x RADIUS x STRAIN PER UNIT PRESSURE x SETTLEMENT FACTOR
μ = POISSON'S RATIO
FOR Z GREATER THAN 7.0, F=1.5/Z

Figure 10.—Settlement factors on axis below vertical stress over circular area at surface.

20-foot depth. The net product, depth times factor, is then

$$0.001(10\times1{,}270-20\times525)=2.2.$$

As the strain per unit pressure is the reciprocal of the modulus, the displacement is

$$0.01\times4\times2.2=0.088 \text{ foot.}$$

In figures 1, 2, 6, and 7 a uniform stress has been assumed. A rigid footing concentrates stress at the edges unless the materials there cannot carry the stress. For footings on sand at a shallow depth, the stress is concentrated near the center (parabolic distribution). The effects of these differences on stress and settlement decrease with depth for load on a circular area (*4*), as illustrated in figures 9 and 10.

Effects of Blankets and Preloading

Figure 11 shows the vertical pressure in a homogeneous material under a typical footing, 6 feet square, uniformly loaded to 3 k.s.f. Depth zero is at the bottom of the footing, which may be below the surface; the effect of depth of footing on stress distribution is neglected. Excavation over only the footing area is subtracted from the total footing pressure to obtain the net footing pressure, 3 k.s.f., as illustrated. Consider three layers in the soil foundation having compressibilities of: m_1 for 0 to 6 feet, m_2 for 6 to 18 feet, and m_3 below 18 feet. The compressibility is strain divided by stress for the appropriate range of stress; it is the reciprocal of the modulus of elasticity. A small m corresponds to low compressibility (a high modulus of elasticity).

If m_1 is smaller than m_2, the stresses in the second layer would actually be less than shown in figure 11. The stress distributing ability is roughly proportional to the cube root of $1\div m$ (*5*). Conversely, if m_3 is very small compared to m_2, the stresses will be higher; over a rigid base, the vertical stress at any depth is increased to approximately that at three-fourths the vertical stress at the same depth in a homogeneous material. For simplicity, these effects are not considered in this article.

If the upper layer is a blanket having a very small m_1, the settlement is proportional to m_2 times the area to the left of the stress curve between 6 and 18 feet, plus m_3 times the area below 18 feet. If, over a large area, an excavation equivalent to 150 p.s.f. (0.15) k.s.f. has been made, the area to the left of this stress only produces settlement proportional to the reloading compressibility, which is usually much less than the compressibility for the first application of a load. The principal settlement would be proportional to the area in figure 11 between the vertical line at 0.15 k.s.f. and the stress curve between 6 and 18 feet, as the compressibility above 6 feet was assumed to be small and the material below 18 feet is being reloaded. Using the stress curves for zero lateral distance in figures 1 and 2 and the geometry described, settlement factors for different preloadings were calculated.

The reduction of settlement factors with preloading (assuming 100 percent consolidation with respect to time) for a loaded square area is shown in figure 12 and for a loaded rectangular area, the length of which is equal to 10 times the width, in figure 13. Reading from figure 12, the settlement factor for a uniformly loaded square area on a homogeneous foundation (no preload) is 1.12 at zero depth. For negligible compressibility such as a compacted granular blanket in the top 6 feet under a 6- by 6-foot-square loaded area (fig. 11), the settlement factor is reduced to 0.42 (depth divided by width=6 divided by 6=1). For a preload of 0.15 k.s.f., and a footing pressure of 3 k.s.f., preload divided by applied pressure=0.15 divided by 3=0.05, and the settlement factor below 6 feet, from figure 12, is 0.17. For this preload, the settlement factor is zero below a depth divided

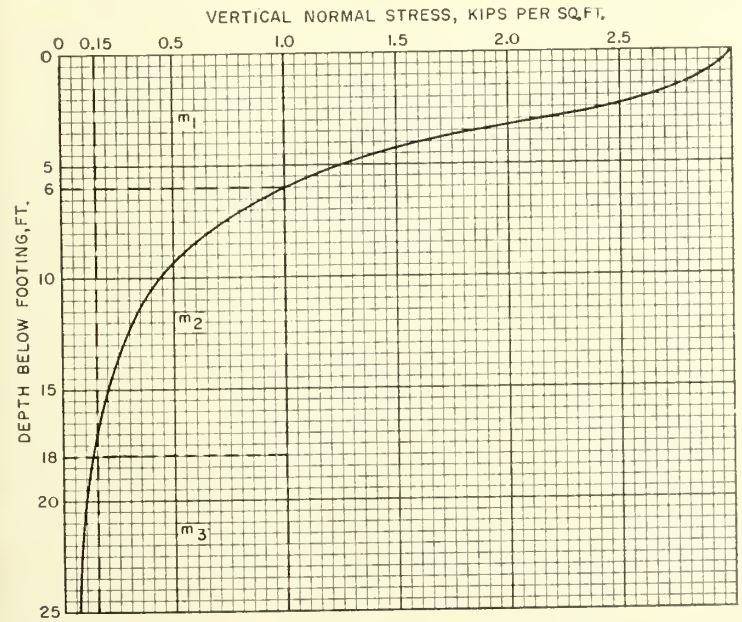


Figure 11.—Stress versus depth under pressure of 3 kips per square foot on a 6-by-6-foot-square footing.

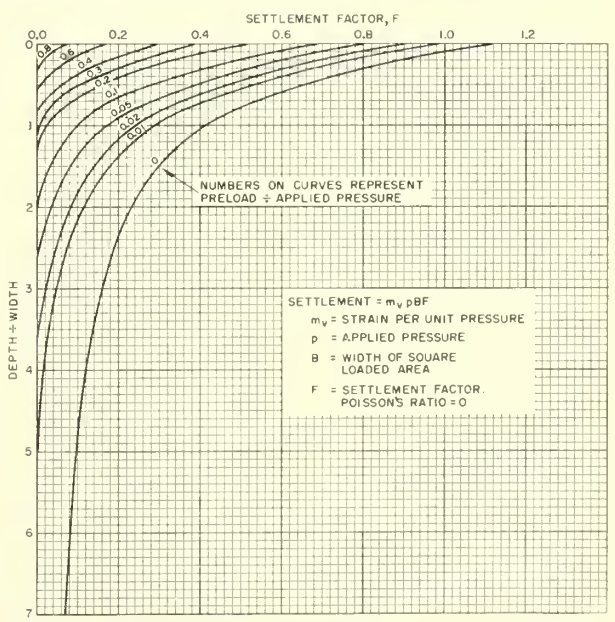


Figure 12.—Maximum settlement under uniformly loaded square of different preloads.

by width=3, which corresponds to a depth of $3 \times 6 = 18$ feet.

To calculate the stress for no settlement (except recompression) below 6 feet, figure 12 data can be used. For depth divided by width=1, preloading of 0.33 multiplied by the applied pressure equals the stress. For the applied load of 3 k.s.f., the preloading is 1 k.s.f. or stress at the depth of 6 feet. The stress could also be determined by use of figure 1, where vertical normal stress divided by applied pressure is 0.33 when depth divided by width=1.

Preconsolidation Factors

Because of the imperfect elasticity of soil, the second application of a stress produces less strain than the first application. The maximum previous stress is termed the preconsolidation pressure. The preconsolidation pressure may be determined from loading history, including water pressure, and may be inferred from the shape of the stress-strain curve in consolidation. The shape of the stress-void ratio curve gives the same result but requires unnecessary arithmetic for calculating void ratio from measured strain. The strain-log stress curve for a sediment is often fairly linear for loads heavier than the preconsolidation pressure.

Care must be taken during sampling and testing to assure that samples are sufficiently undisturbed and that variation in the stress-strain relationship actually is the result of preconsolidation. Figure 14 shows the strain-stress log curve for a residual soil derived from schist. At first glance, it has a preconsolidation pressure between 1 and 2 k.s.f. However, this is simply a function of the plotting scales as shown by the arithmetic plot in figure 15, which reflects a perfectly linear relation between stress and strain.

For some soils, such as mudflows and loess, the strain may be markedly increased by submergence under water.

Compacting the soil by rolling in layers produces an effect similar to preconsolidation. For fine-grained soils, such preconsolidation is about 2 to 4 k.s.f. at 100 percent of maximum density and 1 to 2 k.s.f. at 95 percent of maximum density as determined by AASHO Designation: T 99. The effective preconsolidation is higher for granular materials. The compressibility varies with moisture as well as with density and should be determined for each situation for the range of anticipated conditions.

Time Requirements

If preconsolidation is to be obtained by a temporary preload, the time required for consolidation must be considered. For a deep compressible layer that has drainage at its upper boundary, figure 16 relates the time to consolidation at different depths. Mathematically this is similar to the rate at which a change in surface temperature penetrates the earth (*6*).

For example, use figure 11 and assume that it is desired to preconsolidate the soil at a depth of 18 feet by an additional 0.15 k.s.f., the stress that is to be transmitted from the proposed footing. Assume a temporary preload of 1 k.s.f. over a large area and drainage at and above the 6-foot depth. The time for consolidation or load transfer equivalent to 0.15 k.s.f. at the 18-foot depth may be determined from figure 16. Degree of consolidation may be considered as settlement at a given time divided by final settlement or it may be considered as effective stress increase at a given time divided by the final effective stress increase; that is, when the excess pore pressure from the applied load is zero. The second definition, based on load transfer to the soil skelton, is used herein.

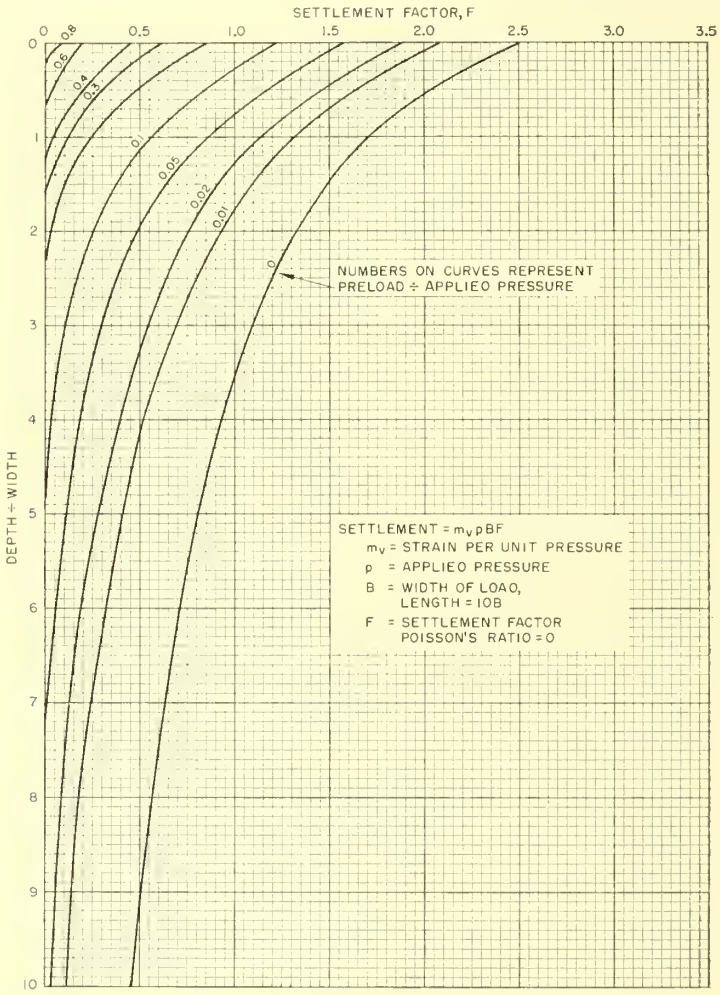


Figure 13.—Maximum settlement under normally loaded rectangle of different preloads.

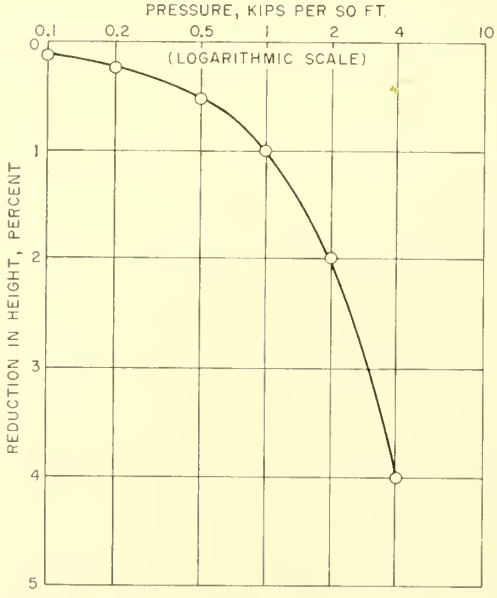


Figure 11.—Shape of stress-strain curves, logarithmic scale.

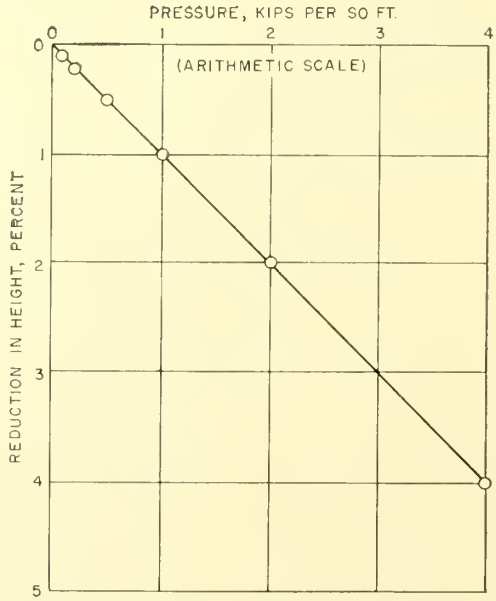


Figure 15.—Shape of stress-strain curves, arithmetic scale.

The depth from drainage surface is

$$18-6-12 \text{ feet}=\mathbf{H}.$$

The required degree of consolidation is 0.15 k.s.f. divided by 1 k.s.f.=0.15. For these coordinates (0.15, **H**), figure 16 gives a relative time of

$$T=tc_v \div \mathbf{H}^2=0.24.$$

The actual time, t, depends on the coefficient of consolidation, c_v, of the soil, which may be determined from laboratory time-consolidation tests on undisturbed samples (7). The time is

$$T\mathbf{H}^2 \div c_v=0.24\times 12^2 \div c_v=34 \div c_v$$

If c_v were 0.1 foot squared per day, the time would be 340 days. A comparison of the curves in figures 16 and 11 shows that degree of consolidation decreases more rapidly with depth than does stress. Because of this, with respect to footing stress the soil between depths of 6 feet and 18 feet will be overconsolidated and below 18 feet will be underconsolidated. Much more time would be required to fully preconsolidate the soil below the 18-foot depth.

Preload Duration

It is sometimes considered that 100-percent consolidation can be obtained by using a balanced combination of preload and preload duration. For example, a preload equal to twice the design load and a time sufficient to obtain 50-percent consolidation under the preload would be a balanced combination. Such a combination would satisfactorily prevent additional consolidation under the design load, if the preloading consolidated all points within the compressible layer to at least 50 percent of the preload stress. But care must be taken in using average consolidation to determine preload duration; some points will be overconsolidated and others underconsolidated in relation to design load stresses. For example, by using table 1 data, for a layer having two drainage faces, both faces permeable, and an average degree of consolidation of 0.5, the time factor (T) equals 0.05. For this same time factor, table 3 contains an interpolated degree of consolidation for a point at the center of the layer of only 0.23 (between 0.046 and 0.060 opposite center of slab). This point is represented by point **B** in figure 17. Thus from a temporary load twice the final load, the average degree of consolidation would be $0.5\times 2=1$ or 100 percent, but a central point would be consolidated to only (0.23×2) or 0.46 percent of the final load.

The consolidation remaining is represented in figure 17 by the area **ABC**. This area has been evaluated as 20 percent of the total load transfer of the final load. The remaining settlement then would be approximately 20 percent of the settlement under the final load. For example, if settlement under the final load were 1 foot, then a 20-percent increase in load transfer would increase the settlement to 1.2 feet. This would be partly compensated by rebound of the overconsolidated material adjacent to the drainage faces. Table 2 shows the consolidation remaining for different preloads and balanced durations in relation to average consolidation. To obtain complete

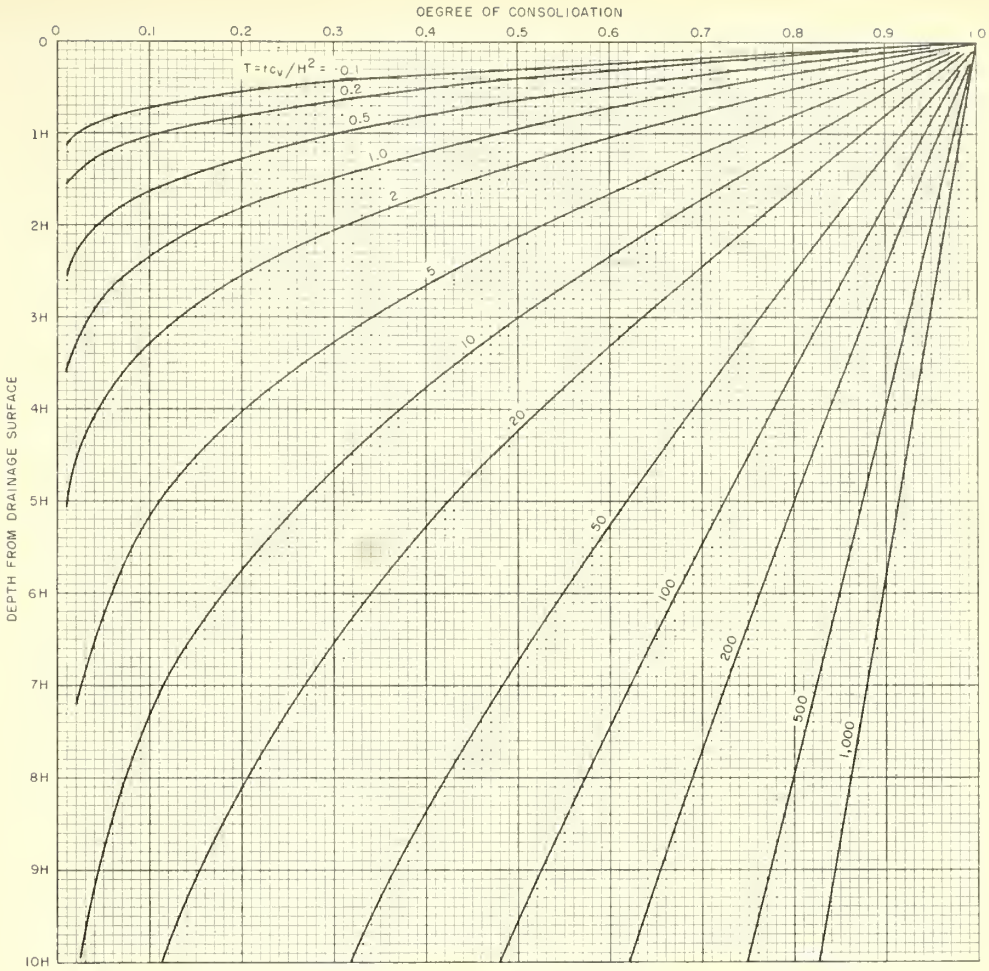


Figure 16.—Time-consolidation below surface of semi-infinite mass.

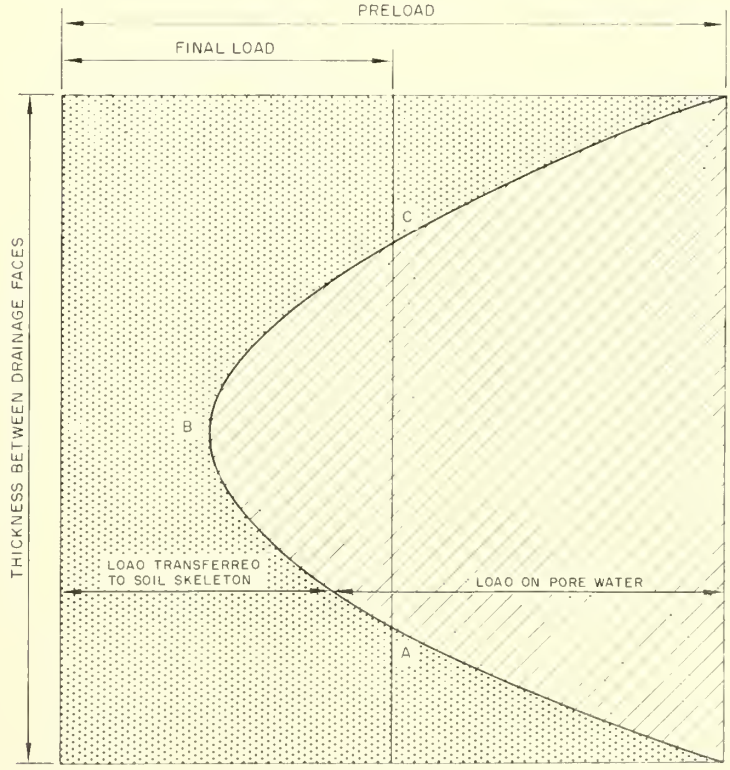


Figure 17.—Load transfer during preloading of layer having two drainage faces.

Table 1.—Average consolidation of uniformly stressed mass

Boundary conditions	Time factor,[2/] $T = \frac{C_v t}{H^2}$, for degree of consolidation or load transfer of--										
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	0.95
Slab of thickness H:											
Both faces permeable	0.0005	0.002	0.008	0.018	0.031	0.05	0.07	0.10	0.14	0.21	0.28
One face impermeable:											
Ratio (pressure at permeable face to pressure at impermeable face):											
0	0.024	0.049	0.100	0.154	0.217	0.29	0.38	0.50	0.66	0.95	1.22
0.2	0.009	0.027	0.073	0.126	0.186	0.26	0.35	0.46	0.63	0.92	1.19
0.4	0.005	0.016	0.056	0.106	0.164	0.24	0.33	0.44	0.60	0.90	1.17
0.6	0.003	0.012	0.042	0.092	0.148	0.22	0.31	0.42	0.58	0.88	1.15
0.8	0.002	0.010	0.036	0.079	0.134	0.20	0.29	0.41	0.57	0.86	1.14
1.0	0.002	0.008	0.031	0.071	0.126	0.20	0.29	0.40	0.56	0.85	1.13
1.5	0.002	0.006	0.024	0.058	0.107	0.17	0.26	0.38	0.54	0.83	1.11
2.0	0.002	0.005	0.019	0.050	0.095	0.16	0.24	0.36	0.52	0.81	1.10
5.0	0.001	0.003	0.013	0.034	0.069	0.12	0.20	0.32	0.48	0.77	1.07
10.0	0.001	0.003	0.011	0.028	0.060	0.11	0.18	0.30	0.46	0.75	1.05
Infinite	0.001	0.002	0.009	0.024	0.048	0.09	0.16	0.28	0.44	0.73	1.03
Sphere of diameter H	0.00005	0.0002	0.001	0.002	0.005	0.008	0.012	0.019	0.028	0.046	0.063
Cylinder of diameter H	0.0002	0.001	0.002	0.005	0.010	0.016	0.024	0.036	0.053	0.083	0.114
Cylinder of diameter H, having internal drain of diameter d; ratio of:											
0.01	0.021	0.046	0.104	0.167	0.24	0.33	0.44	0.58	0.78	1.10	1.45
0.025	0.014	0.032	0.075	0.124	0.18	0.25	0.33	0.44	0.58	0.86	1.13
0.1	0.005	0.014	0.037	0.064	0.096	0.132	0.178	0.24	0.32	0.46	0.60
0.2	0.002	0.006	0.019	0.035	0.054	0.077	0.105	0.14	0.19	0.28	0.37
Steady accumulation on impervious base to depth H, in time t	0.05	0.10	0.20	0.31	0.45	0.62	0.90	1.3	2.2	4.3	10
Steady load increase to time t	0.002	0.005	0.017	0.039	0.071	0.11	0.17	0.26	0.41	0.83	1.67

1/ Unless otherwise noted, load is assumed to be applied instantaneously when time, t, equals zero.

2/ c_v = coefficient of consolidation; t = time; H = distance based on boundary conditions.

consolidation, in the example, the preload would have to be equal to 1 divided by 0.23 or 4.3 times the final load. As an alternate the preload could be left unchanged and the time, *T*, increased to 0.095 (table 3) for 50-percent consolidation.

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(*2*) *Stress Distribution in a Homogeneous Soil*, by C. R. Foster and S. M. Fergus, Highway Research Board Report No. 12–F, 1951.

(*3*) *Influence Charts for Computation of Stresses in Elastic Foundations*, by Nathan M. Newmark, University of Illinois, Engineering Experimental Station, Bulletin No. 338, vol. 40, No. 12, Nov. 10, 1942.

(*4*) *Application of Triaxial Compression Test Results to Highway Soil Problems*, by Edward S. Barber and C. L. Sawyer, Triaxial Testing of Soils and Bituminous Mixtures, ASTM Special Technical Publication No. 106, October 1951, pp. 228–247.

(*5*) *Triaxial Compression Test Results Applied to Flexible Pavement Design*, by Edward S. Barber, Public Roads, vol. 25, No. 1, September 1947, pp. 1–8.

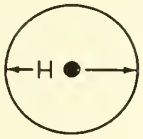
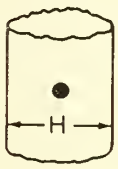
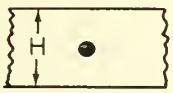
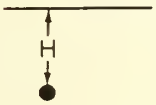
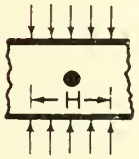
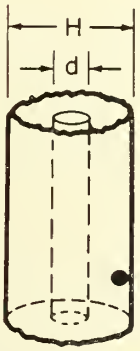
(*6*) *Conduction of Heat in Solids*, by Horatio Scott Carslaw and J. C. Jaeger, Oxford Press, 1947.

(*7*) *Observed Settlements of Highway Structures Due to Consolidation of Alluvial Clays*, by Edward S. Barber, Public Roads, vol. 26, No. 11, December 1951, pp. 217–223.

Table 2.—Consolidation remaining after temporary preloading

Preload	Balanced consolidation	Average consolidation remaining
Percent of final load	Percent of preload	Percent of final load
110	91	2
120	83	4
150	67	10
200	50	20
300	33	35
400	25	45
500	20	50

Table 3.—Consolidation at a point in a uniformly stressed[1] mass

Boundary conditions[2] and location point	Time factor,[3] $T = \frac{C_v t}{H^2}$, for degree of consolidation or load transfer of--										
	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	0.95
 Center of sphere---	0.014	0.016	0.021	0.025	0.030	0.035	0.040	0.048	0.058	0.076	0.094
 Center of cylinder-	0.017	0.021	0.028	0.035	0.042	0.050	0.060	0.073	0.090	0.120	0.150
 Center of slab-----	0.025	0.033	0.046	0.060	0.076	0.095	0.117	0.146	0.187	0.256	0.328
Impervious edge of slab----------	0.100	0.130	0.185	0.241	0.305	0.379	0.467	0.585	0.747	1.022	1.312
 Below surface of semiinfinite mass-	0.130	0.185	0.304	0.465	0.705	1.10	1.82	3.37	7.79	31.7	127
 Center of slab with impervious boundaries--------	0.033	0.046	0.076	0.116	0.176	0.274	0.455	0.842	1.95	7.79	31.7
 Impervious edge of hollow cylinder d/h =											
0.01----------	0.050	0.076	0.132	0.20	0.27	0.36	0.47	0.61	0.81	1.13	1.48
0.025---------	0.041	0.062	0.106	0.156	0.21	0.28	0.36	0.47	0.61	0.89	1.25
0.1----------	0.029	0.040	0.066	0.093	0.13	0.16	0.21	0.27	0.35	0.49	0.63
0.2----------	0.021	0.029	0.045	0.061	0.08	0.10	0.13	0.17	0.22	0.31	0.40

1/ Stress assumed to be applied instantaneously when time, t, equals zero.

2/ Boundaries are pervious unless noted by heavy line.

3/ c_v = coefficient of consolidation; t = time; H = distance based on boundary conditions.

An Interpretive Review

Mechanisms of Soil-Lime Stabilization

OFFICE OF RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported by [1] SIDNEY DIAMOND, Highway Research Chemist, and EARL B. KINTER, Highway Research Engineer, Materials Division [2]

The authors of this article present information contradicting some of the hypotheses generally held concerning the mechanisms of the physico-chemical and chemical reactions occurring when lime is added for stabilization of clay soils. The importance of cation exchange, flocculation, and carbonation in the stabilizing process is discounted. Although no test data are included, a summary of results of recent work by the authors is applied to the evaluation of literature available on the mechanisms of the reactions responsible for soil-lime stabilization.

The authors distinguish between rapid ameliorative effects and long-term cementitious reactions in the stabilization process. They postulate that in soil-lime-water mixtures lime is quickly adsorbed by clay mineral surfaces; the ameliorative effects are the result of a very rapid reaction at the edge-to-face points of contact of clay particles within flocs. The reaction is believed to occur between the adsorbed lime on the face surfaces and the alumina and silica groups on the edge surfaces. The reaction involves the production of very small amounts of cementitious compounds such as calcium and aluminum silicate hydrates—which strengthen the contact points in the floc structure sufficiently to improve plasticity and reduce volume change—but not enough to produce the greater strength required for adequately stabilized soil material. The long-term reactions involve the slow and continued formation of further amounts of calcium and aluminum silicate hydrates in the void spaces within and between the flocs. The available literature suggests that a through-solution mechanism is responsible for the long-term reactions, but evidence of the physical adsorption of lime on clay surfaces suggests a surface chemical reaction that is not dependent on prior dissolution of silica and alumina from the clay.

Introduction

STABILIZING soil by the addition of quicklime or hydrated lime is an ancient art that was successfully adapted by engineers of the Texas Highway Department and is now being extensively practiced. The purpose of the authors in writing this article was to attempt to critically evaluate current knowledge of the mechanism or mechanisms responsible for the soil stabilization process, not to discuss the history or the current engineering aspects of soil-lime stabilization. A better understanding of the mechanisms should make it possible to further the development of practical stabilization procedures so that better advantage can be obtained from the use of lime.

The reactions of soil upon treatment with lime are complex, and physical changes in the soil often are dramatic. It has been widely proposed that these unusual changes are the result of: (1) Cation exchange, that is, replacement of the exchangeable sodium, magnesium, or other cation previously held by the soil clay by calcium cations derived from the lime; (2) flocculation of the clay, and consequent increase in effective grain size; (3) carbonation, that is, reaction of the lime with carbon dioxide from the atmosphere to form calcium carbonate, which has been said to exert cementing action; and (4) so-called *pozzolanic reactions* with soil constituents to generate new minerals of a cementitious nature. As explained in detail the authors believe that these four phenomena, although all may occur in a given soil-lime mixture, do not adequately account for the effects observed.

The factor of cation exchange has been mentioned by many authors, yet familiarity with the cation exchange properties of soils should have eliminated this as a serious explanation for the stabilizing effects of lime on soil. Soil scientists know that many natural soils are almost calcium saturated. For example, results of recent research show that the montmorillonitic soils of the southwestern United States (classed as *Grumusols* in soil science classification systems) are normally two-thirds to three-quarters calcium saturated (*1*).[3] Despite this predominance of calcium in the exchange complex, these soils exhibit all the classic deficiencies associated with montmorillonitic soils and, when they are used as subgrades, require stabilization treatment. Although it might be thought that complete calcium saturation is required for stabilization, research results with sodium-saturated soils have demonstrated that when lime is added to a dilute clay suspension in a quantity in excess of that required for saturation a complete exchange of calcium for sodium does not take place (*2*)—not even when conditions are favorable for cation movement. Exchange in comparatively dry compacted soils is undoubtedly less complete.

The concept that flocculation plays a major part in soil-lime stabilization is often voiced, but careful examination of previously known facts shows this also to be an inadequate explanation for the stabilization. Some soils, including most of the red and yellow soils of the southeastern part of the United States, are naturally flocculated; this can be seen by shaking the soil in water and examining the resultant suspension. Despite this natural flocculation these soils are not stable, and they do respond to lime treatment. Furthermore, it is well known that many chemical agents, including different salts, alcohols, acids, ketones, cause immediate flocculation when mixed with clays, but they are valueless for stabilization. The fact that flocculation of clay occurs as a consequence of the addition of lime is a well-known phenomenon, but the achievement of flocculation is clearly not the mechanism by which lime stabilizes soils.

The hypothesis that soil-lime stabilization depends on the carbonation of the lime to form calcium carbonate can be dismissed by reference to the reports of many studies. In these studies reaction of the lime with atmospheric carbon dioxide was precluded by sealing the samples, and the characteristic modification of properties and development of strength associated with lime stabilization were observed. As demonstrated by Eades, Nichols, and Grim (*3*), carbonation does take place in the field; but the gain in strength said to accrue from cementation of soil grains by calcium carbonate has not been conclusively demonstrated. The writers believe that the long-term reaction of uncarbonated lime with the soil itself would far outweigh any such contribution, and carbonation is probably a deleterious rather than a helpful phenomenon in soil stabilization.

Following the rejection of these inadequate hypotheses, other more nearly adequate but still incomplete explanations of the mechanisms of lime-soil interaction were considered. Experience indicates that at least two distinct stages of reaction are involved: (1) the immediate or rapid processes that ameliorate the

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[2] The authors acknowledge the contributions made by Dr. W. C. Ormsby, National Bureau of Standards, and Donald G. Fohs, Bureau of Public Roads, in discussions of the literature of soil-lime stabilization.

[3] References indicated by italic numbers in parentheses are listed on pp. 265 and 273.

water-sensitive properties of untreated clay soil, and (2) the slower, long-term reactions that cause formation of the final cementitious products that are indicated by the gradual development of strength in compacted soil-lime mixtures. These two stages of reaction are discussed separately.

Summary

Among physico-chemical mechanisms commonly suggested as explanations for the stabilizing effects of lime on soil, cation exchange replacement of existing cations by calcium, flocculation, and carbonation have been prominent. Because many soils in need of stabilization are naturally calcium saturated, flocculated, or both, the first two mechanisms can no longer be seriously considered; and as soil-lime systems sealed from contact with carbon dioxide develop the normal indications of stabilized soils, carbonation must also be rejected.

The effects of lime on soils are such that two stages of reaction can be detected: (1) an early stage in which the plastic properties of the soil are greatly improved but little permanent strength is developed; and (2) a subsequent stage marked by the slow development of strength and the accumulation of soil-lime reaction products. Among the effects in the first state are: (1) large increases in the plastic limit, generally followed by a reduction in the plasticity index; (2) a sharp reduction in the apparent content of clay-size particles as they are bound into flocs stable against the dispersion incident to mechanical analysis; (3) increases in the moisture and the compactive effort required to achieve a given density; and (4) a reduction in such parameters as swell pressure, volume change on drying, and permeability. After the addition of lime, these changes occur in periods ranging from minutes to a few hours.

Based on results of recent work, researchers have suggested the existence of a lime fixation point; that is, an amount of lime that must be added to a given soil to produce the maximum early stage effects, but lime that is not available for further reaction. The mechanism of the effect was discussed in terms of an apparent crowding of excess calcium cations onto the clay following the addition of lime. This in turn was tentatively attributed to pH-dependent exchange sites originating at the edges of the clay particles.

The results of recent experimental work by the authors contradict these hypotheses. The cation crowding effect is in reality one of physical adsorption of calcium hydroxide onto the clay surfaces. The authors postulate that the beneficial effects of the addition of lime are caused by the almost immediate but limited chemical reaction at the points of contact between the edges and faces of primary clay particles within the flocs formed by the normal electrolyte effect of added lime. This reaction is visualized as the formation of small amounts of tetracalcium aluminate hydrate by reaction of the exposed $Al(OH)_x$ groups at the edges of the clay particles with lime sorbed on the faces of adjacent particles. This immediate reaction is supplemented by somewhat slower reaction of the silica with lime to generate tobermorite gel.

The nature of the compounds that are considered responsible for the slow development of strength in soil-lime systems has been discussed in some detail. The exact products formed vary according to the kind of clay and the reaction conditions, especially temperature. At least two phases are produced, a calcium silicate hydrate and a calcium aluminate hydrate. The former is usually tobermorite gel; the latter is a well-crystallized hexagonal compound, which is probably an impure (substituted) tetracalcium aluminate hydrate, and is characterized by a 7.6 A. basal spacing independent of drying conditions. At temperatures only slightly above normal room temperature a different calcium aluminate hydrate—the cubic tricalcium aluminate hexahydrate—is produced.

Quartz, mica, and other phases considered less reactive than clays may also react under appropriate conditions and the result will be similar cementitious products. Quaternary phases in which silicon and aluminum atoms occur in distinguishable lattice positions are not commonly formed, but quaternary phases do occur when lime reacts with previously calcined clay or with the amorphous clay mineral allophane.

Detailed knowledge is lacking on the mechanisms of the chemical reactions that produce the final cementitious products. Evidence exists, however, to show that the reactions are favored by conditions of high pH, which would make silica more soluble; addition of sodium hydroxide to lime-clay systems produces significant strength gains at early ages. This implies a mechanism involving reaction of dissolved silica and alumina with calcium ions. Evidence also exists to show that the reaction is preceded by sorption of the calcium hydroxide from solution and this strongly implies a direct surface reaction with the clay. It may be that both mechanisms are operative.

Rapid Ameliorative Effects

Addition of lime to plastic soils reduces the plasticity index, according to reports of many researchers (*4*, *5*, *6*). The separate effects of lime on the plastic and liquid limits of plastic soil are discussed individually. Usually an immediate increase in the plastic limit is observed on the addition of lime. The amount of this increase varies directly with the amount of lime added, up to some limiting lime content; further increments of lime usually bring little or no additional increase. The point of inflection of the plot of lime added in relation to the plastic limit has been designated by the appellation "lime fixation point" (*7*). The implications involved in this term will be discussed later.

The effect of the addition of lime on the liquid limit is not so easy to summarize because of conflicting data in published reports. Some authors report decreases in the liquid limit on addition of lime (*4*, *5*), others report that this parameter may increase substantially (*8*, *9*), and others (*6*, *10*, *11*) report that both increases and decreases occur, depending on the individual soil being tested. These discrepancies require some explanation. It is known that the liquid limit of a clay is far more sensitive to the kind of cation present than the plastic limit. Calcium-saturated clays have substantially lower liquid limits than the same clays saturated with sodium or certain other cations (*12*). As some—although not complete—cation exchange occurs on addition of lime, the effect of the lime may vary and depends on the extent to which preexisting cations other than calcium are exchanged. Studies on clays of unknown original cation status consequently are difficult to interpret.

Clare and Cruchley (*8*) report data that showed a drastic increase in the liquid limit for a particular clay upon the addition of lime; this clay was almost completely calcium saturated in its natural state. They also state that these data are typical for the other clays tested, in which the original calcium saturation ranged from 35 percent upward. The present writers, and many others, have observed that mixtures of lime and calcium-saturated clay, stored so as to prevent evaporation or carbonation, assume an increasingly dry appearance; sometimes reaction is so extreme that a free-flowing slurry mixture may stiffen so that it will no longer pour or it will not take the shape of the container. This reaction would be reflected by an increase in the liquid limit of the soil. Generally the liquid limit seems to increase on the addition of lime when there is no strong specific tendency in the opposite direction created by cation exchange effects.

When the liquid limit increases on the addition of lime to the soil, the increase usually is not as large as the accompanying increase in the plastic limit. Thus the separate effects of the addition of lime to the soil on the liquid and plastic limits usually combine to result in a rather sharp decrease in the plasticity index. Usually only small percentages of lime are required to produce this sharp decrease in plasticity index, and additional increments of lime are relatively ineffectual. Sometimes, however, addition of the extra increments of lime reverses the trend and produces increases in the plasticity index. Because the plasticity index is a composite parameter an additional complication arises. The plasticity index may not accurately reflect some of the real changes in the system; for example, equal increases in the liquid and plastic limits following a given treatment would be reflected as no change in the plasticity index.

Additional changes generally accompany the aging of the system. Sometimes this aging effect is not large. Lund and Ramsey (*6*), for example, reported that very little change occurred in the Atterberg limit values of soil-lime mixtures after the first hour. However, often substantial time-dependent changes do ensue. For example, Wolfe and Allen (*13*) reported substantial increases in plasticity index for lime-soil mixtures cured for 2 days, as compared with mixtures tested immediately after the addition of lime. When

the curing periods were extended from 2 days to a period of 7 to 28 days, the effect was reversed, and significant decreases in plasticity index were recorded; a nonplastic condition was recorded for most of the lime-soil mixtures tested.

Grain-size distribution

Changes in the effective grain-size distribution occur almost immediately following the addition of lime to a clay soil. Data obtained by conventional sieve and hydrometer analysis show that a major decrease in the content of clay-size particles occurs within the first hour; subsequent results obtained after periods ranging from 1 hour up to 240 days show only small additional changes (*6*). The new grains produced as a result of the lime treatment are mostly sand size and, although relatively weakly bonded, are for the most part capable of withstanding a 5-minute dispersion period in a mechanical mixer (*6*, *13*).

Moisture-density relations

Many researchers have reported that the density to which a soil can be compacted at a given moisture content is usually reduced significantly when lime is added, and delay in compaction causes a further reduction in density. To achieve maximum density for a given compactive effort the required moisture content usually increases, sometimes rather significantly. However, according to results summarized by Herrin and Mitchell (*14*), lime in excess of a relatively limited amount, on the order of 5 percent by weight of soil, generally causes little additional increase in the optimum moisture requirement.

Volume change effects

Lund and Ramsey (*6*) reported a drastic reduction in the volume change. This effect was observed within the first hour after the addition of lime, and the soil test showed that about 3 percent of lime was sufficient to obtain the maximum volume change effect. A corresponding immediate increase occurred in the shrinkage limit itself. Similar results were reported by Wolfe and Allen (*13*). Mitchell and Hooper (*15*) reported that lime, specifically dolomitic lime, markedly reduced the swelling of specimens tested under a modest surcharge pressure. The effect was observed after an aging period of 24 hours. A reduction in expansion pressure on addition of lime was observed by Wolfe and Allen for some soils, but not for others (*13*).

Clare and Cruchley (*8*) reported significant increases in the amount of moisture held against a given suction after the addition of lime.

Permeability

Although few data are available, the permeability of compacted soil-lime mixtures has been reported to be much less than that of compacted soil alone (*13*).

These different short-term effects can be regarded as different aspects of the ameliorative effect that lime has on the properties of clay soils. The authors attempt to define herein exactly what chemical or physico-chemical action is being reflected in these sometimes drastic changes. Considerable evidence seems to point to the conclusion that these rapid responses are not the result of the pozzolanic reactions to which permanent strength gains are attributed. It has been well documented that the development of strength of specimens, particularly after their having been soaked, is a fair index of the amount of cementitious compounds formed (*16*). The experience of many researchers summarized by Herrin and Mitchell (*14*) and the results documented by Anday (*17*) indicate that strength development in soil-lime systems is a comparatively slow process that characteristically requires from several weeks to many months at normal temperatures.

Mitchell and Hooper (*15*) pointed out that, if significant amounts of lime were consumed by permanent cementing reactions at early ages, delay between mixing and compaction should adversely affect the final strength of the soil. But, when all of their lime-soil samples were compacted to the same density, no adverse effect on the final strength was recorded for delays of up to a day between mixing and compacting. Similarly, if significant permanent cementing were to occur at early ages, subsequent remolding of the specimen should bring about a strong reduction in the ultimate strength developed.

Data developed by F. D. Shepard in 1963 in a study made at the laboratories of the Virginia Council of Highway Investigation and Research, Charlottesville, Va., showed that remolding soil-lime mixtures as long as a week after initial compaction caused no deleterious effect on the ultimate strength. But Shepard's data also show that when soil-cement specimens, in which cementation occurs rapidly because of hydration, were remolded after a week of curing, their ultimate strength was markedly reduced. Thus, extensive development of pozzolanic reaction products seems also to be ruled out as a mechanism responsible for the rapid improvement of the properties of plastic soils when lime is added.

Lime Fixation Point

Extensive investigations undertaken at Iowa State University (*7*, *18*, *19*, *20*, *21*, *22*) have led to the concept of the lime fixation point. This point was defined as one at which the percentage of lime is such that additional increments of lime produce no appreciable increase in the plastic limit. It was hypothesized that excess calcium cations, derived from the lime, in some fashion crowd onto the clay particles and cause them to become electrically attracted—a process causing flocculation in which weak bonds exist between the flocs. Additional lime, which produces calcium cations in excess of those that can crowd onto the clay, produces no further change in the plastic limit. Calcium held by the clay in amounts up to the lime fixation point was considered to be immune to further reaction with the clay to form cementitious compounds. The sedimentation velocity of flocs formed by adding lime to a clay suspension reached a maximum near the lime fixation point, indicating that floc size was at a maximum at this point (*20*).

Ho and Handy (*21*) cited evidence that calcium was retained by bentonite from lime-bentonite slurries after a very limited washing treatment. The amount of calcium retained increased as the percentage of lime in the slurry was increased, and for modest percentages of lime the retained calcium was far more than the normal cation-exchange capacity of the clay. These authors (*21*) hypothesized that the calcium in excess of the normal cation-exchange capacity was held at new exchange sites generated at the edges of clay particles by the increasing dissociation of acidic (——Si(OH)$_x$) groups as the pH increased; hence the term pH-dependent exchange. However, they noted, without explanation, that the amount of calcium retained continued to increase steadily as increments of lime were added far beyond the lime fixation point and no break in the curve was evident.

Ho and Handy (*21*) also studied several other features of the lime-bentonite system. Small additions of lime (up to about 2.2 percent by weight of clay) increased the relative viscosity of calcium bentonite slurries; however, as aging of the soil samples increased, a continual increase was required in the lime content to attain maximum relative viscosity. These authors also made differential thermal studies of dried lime-bentonite slurries, and stated that samples to which only a small amount of lime had been added failed to generate the normal endothermic response characteristic of the lime.

Ho and Handy (*22*) also reported the results of measurements of the zeta potential—a quantity related to the net negative or positive charge carried by the clay particles in dilute suspension—of lime-treated bentonite aged either for several days (fresh) or for a year (aged). Calcium-saturated bentonite had a zeta potential of about −20 millivolts. Small additions of lime caused a slight increase in this potential, and larger additions generated a modest decrease that was identical for both fresh and aged samples. Sodium-saturated bentonite had a zeta potential of about −40 millivolts; additions of lime to this clay up to about 6 percent caused strong proportional reductions to −26 millivolts for the fresh samples and −22 millivolts for the aged samples, and larger additions of lime caused no further change. The authors of this article do not agree with Ho and Handy's interpretation of their results but attribute the large effect reported for the sodium bentonite to the ordinary process of cation exchange of calcium for sodium. This exchange, in accord with the results reported earlier by Prikryl and Esterka (*2*), was not entirely complete.

The authors are not satisfied that the interpretations of the early ameliorative effects of lime on clay suggested by the Iowa State University researchers could be correct in detail. The apparently unlimited crowding of positively charged cations onto the limited external surface of the montmorillonite seems to be an unlikely phenomenon; if it were to occur, a large decrease and eventual reversal of the negative charge of the clay would be

expected to take place because of the limited number of ——Si(OH)$_x$ groups on the edges of the clay particles that could dissociate to generate negative charges to balance the cation uptake. A series of investigations was therefore carried out in an attempt to obtain more information about these matters. Details of these investigations are not presented here, but some of the results have been summarized.

Results of Investigations

Measurements were carried out on Wyoming bentonite, which had been saturated with calcium cations by standard laboratory procedures and checked for completeness of saturation. Small quantities of the calcium-saturated clay were then shaken with a large amount of saturated calcium hydroxide solution for different periods of time at room temperature, and the clay was separated by centrifugation. The supernatant solutions were recovered and promptly analyzed for calcium content by versenate titration and for hydroxyl content by titration against standard acid. The results listed in the following paragraphs were obtained

- A very rapid reduction occurred in the concentration of both calcium and hydroxyl ions in the solution.
- The proportion of the ions remaining in solution was stoichiometric, thus indicating that equivalent proportions of hydroxyl and calcium ions—two hydroxyl ions for each calcium ion—had been taken up by the clay.
- About 3 percent of lime by weight of the clay was adsorbed within the first 5 minutes, the minimum time in which it was possible to perform the manipulations. This amount corresponds roughly to the lime fixation point of the Iowa State researchers.
- Further sorption continued at a declining rate for the 3 weeks that the experiments were continued. At that time, the total amount of lime removed from the solution amounted to about 20 percent of the weight of the clay.
- When the centrifugally separated clay was washed with an amount of distilled water equal to the volume of the saturated lime solution originally used, essentially complete removal of the adsorbed lime was obtained if the washing was done shortly after the onset of the experiment. However, the ratio of hydroxyl to calcium ions removed was slightly less than stoichiometric.
- After a sample had been in contact with the lime solution for several days and had adsorbed considerable additional lime, a single wash, as outline in the preceding paragraph, removed only part of the lime; subsequent washings removed smaller and smaller amounts of lime. The ratio of hydroxyl to calcium ions removed in successive washings declined from values fairly close to stoichiometric (about 1.7) to about 1.
- In differential thermal analysis (DTA) of dried clay, which had sorbed about 4 percent of lime from solution in about an hour of contact, no lime was detected. A mechanical mixture of dry clay containing the same amount of dry lime produced a readily detectable endothermic response at 500° C., which is a response characteristic of crystalline lime.
- No peaks for crystalline calcium hydroxide were observed on the X-ray diffraction pattern of the clay recovered from the lime solution; the corresponding dry mixture produced easily detectable peaks under the same diffraction conditions.
- Measurements of surface charge of the calcium bentonite were carried out on untreated clay and on clay recovered after its immersion in saturated lime solution for an hour. The method used was the one Pike and Hubbard used for a similar determination of the surface charge of hydrating cement particles (*23*). No significant change in negative charge was recorded.

In other experiments, lime was added to suspensions of calcium-saturated bentonite of restricted water content. At lime contents of 4 percent or less by weight of clay, the electrical conductivity of the suspension decreased to very low values in less than 24 hours. This was regarded as an indication that the lime was being rapidly adsorbed, accompanied by a consequent decrease in concentration of the lime electrolyte in the pore solution. Removal of some of the pore solution by high-speed centrifugation and subsequent chemical analysis confirmed this interpretation. The response to lime in clay-water systems of restricted water content was similar to that occurring in dilute suspensions.

There was no evidence of heat production or heat removal accompanying the addition of lime to calcium bentonite suspensions of restricted water content, in tests carried out over several hours with a moderately sensitive calorimeter. This finding is considered to be further indication that the lime sorption process is one of physical adsorption rather than chemical reaction.

Interpretations

The results on the early soil-lime reactions have been interpreted by the authors somewhat differently than has been reported by others. The evidence cited permits the following interpretations:

- Calcium hydroxide is physically adsorbed from solution at a very rapid rate by calcium-saturated clay and, presumably, also by clay saturated by other cations. This adsorption is largely reversible at very early stages of reaction, but adsorption is soon followed by a reaction that produces calcium silicate hydrates. The conclusion that lime is physically adsorbed on clay surfaces was also reached by Prikryl and Esterka (*2*).
- The adsorption removes calcium ions and hydroxyl ions from solution concurrently and does not reflect a crowding of cations only onto new exchange sites generated at high pH levels, as previously postulated by others.
- As both ions are sorbed in equivalent amounts (within the limits of the accuracy of the analysis), no significant change occurs in the net negative charge of the clay particles. This does not preclude small changes in the zeta potential after chemical reaction has proceeded for some period of time.
- The amount of calcium and hydroxyl ions sorbed immediately—about 3 percent by weight of clay—would correspond roughly to sorption of a little more than a monomolecular layer of calcium hydroxide on the external surfaces of the clay. This sorption corresponds roughly to the lime fixation point, and it is suggested therefore that the latter represents approximately a monolayer of lime on the external surfaces of the particular clay.
- The very large amount of slow sorption beyond the amount rapidly sorbed is believed by the authors to reflect several additional processes, but principally the process of slow reaction of adsorbed lime with the clay surfaces to produce calcium silicate and calcium aluminate hydrates. The authors also believe that, at least in wet systems, these products spall from the clay surface, thus liberating fresh clay surface for further adsorption and reaction. Physical adsorption of lime onto the newly formed reaction products is also likely. Finally, a slow, restricted entry of lime into the interlayer spaces of the clay may take place.

Mechanisms of Rapid Reaction

The observation that adsorption of lime occurs in clay-lime systems does not in itself provide any particular indication of how the clay properties are so drastically changed within a short time after the addition of lime. The mechanics of this process require additional explanation and clarification.

Strong flocculation is commonly observed when lime is added to clay. In general, according to current colloid-chemical concepts (*24*), a clay flocculates on addition of an electrolyte because of the modifying effect of the electrolyte on the extension of the electrical double layer from the surfaces of the clay particles. The electrolyte represses the double layer and thus reduces the electrostatic repulsive forces between clay particles. The result is a net attraction, especially between negatively charged faces and positively charged edges of adjacent particles, and a cardhouse or double-T structure develops. If the electrolyte is removed from the pore solution, the double layer again spreads out, the repulsive forces between particles increase, the flocs weaken and are reduced in size, and eventually the system deflocculates.

In the experiments partially reported here, removal of lime by adsorption from the pore solution after a few hours caused very low concentrations of electrolyte; yet the initially formed flocs persisted and became, if anything, more pronounced and stable with lapse of time. Obviously, the lime has more profoundly altered the properties of the clay than can be explained by the flocculation effects of electrolytes described here.

One of the authors previously has demonstrated (*25*) that lime reacts almost instantaneously with hydrous alumina of high-surface area to generate the well-crystallized compound tetracalcium aluminate hydrate, C_4AH_{13}. (In the shorthand notation commonly used for these compounds: C=CaO, A=Al_2O_3, H=H_2O, S=SiO_2.) This compound is also probably formed on the hydration of C_3A in portland cement (*26*). In this article the

authors postulate that in lime-clay systems a similar immediate reaction takes place between the alumina-bearing edges of the clay particles and the lime adsorbed on the clay surfaces.

In particular, the postulation is that this reaction occurs at the points of contact between the edges of one particle and the faces of adjacent particles in the cardhouse structure of the flocs. Calcium silicate hydrate (tobermorite gel) probably forms at these points of contact also, but slowly, perhaps over a period of some hours. Formation of very small quantities of these cementing products at the points of contact is believed to be sufficient to stabilize the flocs and knit the particles together so that plasticity, shrinking and swelling, and other normal clay-water interactions are distinctly inhibited. The individual particles are cemented together well enough within the flocs to resist dispersion, and the flocs may act as single grains in mechanical analysis (*6*). However, the flocs were not bonded to each other well enough to provide significant strength in the overall clay mass, and thus the clay has been ameliorated but not really stabilized.

To develop such a stabilized material capable of holding together and resisting applied loads, compaction is required to obtain a minimum volume of voids, and a sufficient length of time must be permitted for the slow continuing chemical reaction to develop enough additional cementing products to fill the voids at least partially. As shown by Jambor (*16*) the strength developed in lime-pozzolan systems depends to some extent on the kind of cementing agent formed, but also in great degree on the proportion of void space occupied by the cementing agent—the *gel-space ratio* familiar in portland cement technology.

The formation of calcium aluminate and calcium silicate hydrates at particle contacts very early in the reaction process cannot be easily demonstrated because of the very small amounts of these products involved and the nature of the system. The concept is offered as a working hypothesis consistent with the known properties of the system.

Products of Long-Term Reaction

The reactions that occur over a long period of time and are, in the last analysis, responsible for the stabilization of the soil, are of at least as much concern as the ameliorative responses just described. Most soils consist largely of uncombined silica and of silicates of many kinds; alumino-silicates usually predominate in the clay fraction. Consequently, it is not surprising that reaction with lime produces compounds largely of two classes: hydrated calcium silicates and hydrated calcium aluminates.

Except when formed under hydrothermal conditions, the calcium silicate hydrates are invariably poorly crystallized and difficult to detect. Three such phases are known in the literature: tobermorite gel—also called C-S-H gel—and the phases known as C-S-H(I) and C-S-H(II). The gel is a high-calcium phase normally generated in and responsibile for the strength of portland cement concrete. Of the three phases, C-S-H (I) has a distinctly lower calcium content, a somewhat different morphology, and can be distinguished by a very strong exothermic peak that occurs at about 850° C. on DTA. Although C-S-H(I) may be synthesized in a fairly well-crystallized form in the laboratory, the gel phase is invariably much less well crystallized, and it has a maximum of only three X-ray diffraction peaks. C-S-H(II) is a high-calcium phase, whose exact properties are open to some doubt. All of these compounds are presumed to have a layer structure similar to, but not necessarily identical to, that of the well-crystallized mineral tobermorite. The latter may be synthesized readily by hydrothermal means, but its occurrence in soil-lime reaction products produced under normal atmospheric conditions is extremely unlikely.

There are several types of calcium aluminate hydrates. The one commonly formed at normal temperatures is a member of the tetracalcium aluminate hydrate group. This group comprises many crystalline modifications and partially dehydrated states, as well as phases of the same basic structure, but carbonate groups are incorporated in place of some of the hydroxyls. The form generally produced by clay-lime reactions is most similar to a phase in which the composition is $C_4AH_{12}\cdot CO_2$, and it has a basal spacing of 7.6 A. that is not changed by drying. In contrast, pure C_4AH_{13} has a basal spacing of 7.9 A. when moist, and the spacing diminishes stepwise to about 7.4 A. or less upon drying. It is possible that the soil-lime product may incorporate some silicon in isomorphous substitution for aluminum.

Reactions of lime with clay minerals at slightly elevated temperatures generally produce a very different calcium aluminate hydrate, C_3AH_6. This is a cubic phase having a crystal structure different from that of the tetracalcium aluminate hydrates; it forms preferentially at temperatures above about 30° C., and once formed is stable at room temperature. A recent monograph edited by Taylor (*27*) covers the status of knowledge of both calcium silicate and calcium aluminate hydrates.

Goldberg and Klein (*28*) carried out the first published X-ray study of lime-clay reaction products, but detected only calcium carbonate, which was probably produced during air drying of the sample prior to its being X-rayed. Eades and Grim (*29*) reported that reaction of lime with pure clay minerals at 60° C. caused the formation of new minerals. Kaolinite so treated yielded peaks for a poorly crystallized calcium silicate hydrate of unspecified type, and a peak of 5.1 A. (and others), which the present writers interpret as having been caused by C_3AH_6. Eades and Grim could not detect any new crystalline product of lime-montmorillonite reaction, although it was obvious that reaction had occurred. Illite reacted somewhat, but no positive statement was made as to the nature of the reaction product or products.

Hilt and Davidson (*30*) examined the product of a long-term reaction of lime and montmorillonite at room temperature, and finally identified it as a C_4AH_{13} type of material. Its properties were consistent with the previously mentioned product having a 7.6 A. spacing. The published X-ray diffractometer traces indicated weak peaks that were attributed to pooly crystallized calcium silicate hydrates. Glenn and Handy (*31*) studied the products of reaction between several clay minerals and different forms of lime at room temperature, but their results were not completely interpreted. They reported generally that poorly crystallized calcium silicate hydrates and the 7.6 A. calcium aluminate hydrate were found in the kaolinite systems; in addition, an unknown 12.6 A. compound was formed when dolomitic lime was used. Montmorillonite yielded both the 7.6 A. calcium aluminate hydrate and materials whose diffraction peaks were ascribed by the authors to a more nearly pure C_4AH_{13} phase.

Glenn and Handy observed no reaction of lime with quartz, and little with muscovite or vermiculite. In contrast, in studies of the products formed in field soil stabilization projects, Eades, Nichols, and Grim (*3*) reported that considerable reaction had occurred in quartz- and mica-bearing soils. When examined under the petrographic microscope, the quartz and mica particles had developed fuzzy outlines and visible cementing gel was present, not only as a coating on the grains but also in cracks within the grains. Examination by X-ray diffraction disclosed the presence of calcium silicate hydrates, as well as calcium carbonate. The possibility that calcium aluminate hydrates might also have been formed was not discussed.

Diamond, White, and Dolch (*33*) reported that kaolinite and montmorillonite reacted with lime to produce different products under different reaction conditions. At 60° C. both clays produced calcium silicate hydrate classified as C-S-H(I) and the kaolinite produced the cubic C_3AH_6; no crystalline calcium aluminate compound was formed from montmorillonite. At lower temperatures the products from both clays were considered to be tobermorite gel and the 7.6 A. calcium aluminate hydrate. Small peaks at about 9 A. occurred in the montmorillonite products under both conditions; but they disappeared when the products were washed and no explanation for their presence could be offered. These authors reported that both the tobermorite gel and the 7.6 A. calcium aluminate hydrate product were formed by reaction of lime with mica, illite, and even pyrophyllite. Reaction with quartz generated tobermorite gel even at 60° C.

In a paper titled, *X-Ray Studies of Lime-Bentonite Reaction Products*, G. R. Glenn and R. L. Handy reported further results on long-term studies of reaction products of lime and montmorillonite at room temperature at the 66th annual meeting of the American Ceramic Society held in Chicago, Ill., 1964. In general, the formation of tobermorite gel, C-S-H(I), C_4AH_{13}, and possibly C-S-H(II) were reported. Interestingly, fresh mixtures of clay and lime, as well as mixtures that had been allowed to react at room temperature

for several years, yielded only well-crystallized tobermorite on hydrothermal treatment. The aluminum present in the product was believed to have been incorporated as an isomorphous replacement for silicon within the tobermorite lattice.

In all the studies discussed so far, no quaternary compounds (that is, compounds in the lime-silica-alumina-water system) were reported. Apparently, where separate calcium aluminate phases were not developed, the aluminum present was incorporated isomorphously in the calcium silicate phase. In contrast to these results, Benton (*33*) reported that reactions of lime with calcined kaolinite yielded the quaternary compound gehlenite hydrate, C_2AS_x, also called Stratling's compound. This compound was formed in addition to the more usual products, a poorly crystallized calcium silicate hydrate and the 7.6 A. calcium aluminate hydrate.

In a Japanese study (*34*) the same gehlenite hydrate and the 7.6 A. calcium aluminate hydrate were reported as the result of reactions at normal temperatures between lime and Kanto loam soils, which consist mostly of the amorphous clay mineral allophane. At higher temperatures the same soils produced C_3AH_6 and a hydrogarnet quaternary phase instead of these compounds. When gypsum was also included in the treatment of these soils with lime, excellent stabilization results were obtained: the major cementing agent formed was ettringite or so-called *cement bacillus*, a sulfate-bearing phase formed in hydrating cements.

Cementation Products

The early physico-chemical reactions of lime that produce the ameliorative effects on soil clays, and the nature of the cementing compounds that produce the final cemented product have been discussed, but little has been said about the mechanics of the chemical and structural transformations that generate the final compounds. Very little is known about the details of the reaction processes involved, and most information is speculative in nature. This lack of knowledge is one of the major gaps in understanding the soil-lime system.

Eades and Grim (*29*) have suggested that with kaolinite, ". . . the reaction seems to take place by lime eating into the kaolinite particles around the edges with a new phase forming around a core of kaolinite." Electron micrographs by Diamond, White, and Dolch (*32*) tend to confirm this idea because the edges of residual kaolinite particles were ragged and irregular, as though they had been attacked chemically. The probability that partial exfoliation had taken place also was proposed by Diamond et al; they also called attention to the fact that because two distinct crystalline hydration products were produced, a simple topotactic solid-state mechanism was not a reasonable explanation.

At the 1st annual meeting of the Clay Minerals Society (1964) held in Madison, Wis., R. L. Sloan presented a paper, *Early Reaction Determination in Two Hydroxide-Kaolinite Systems by Electron Microscopy and Diffraction*, in which he reported an electron-microscope study of the effects of treating kaolinite with sodium hydroxide and also with dilute lime in suspensions. He confirmed that under such conditions the primary attack on the kaolinite particles took place at the particle edges. When lime was used, he observed what appeared to be nucleation of a reaction product at or near the edges of the kaolinite particles; however, this product did not seem to be one previously reported as occurring in lime-clay reactions, and the exact significance of this observation is uncertain.

For illite and montmorillonite soils, Eades and Grim concluded that ". . . Following the saturation of the interlayer positions with calcium ions the whole clay mineral structure deteriorates without the formation of substantial new crystalline phases." In contrast, Diamond, White, and Dolch reported that new crystalline phases (tobermorite gel and the 7.6 A. calcium aluminate hydrate) were formed from these two minerals, and even in advanced stages of the reaction when most of the clay had been decomposed, the clay that remained retained its crystallinity almost intact. For example, a 2M illite retained all the X-ray peaks characteristic of this polymorph, and montmorillonite retained its characteristic (060) spacing.

Eades, Nichols, and Grim (*3*) suggested that a pH high enough to dissolve silica is an essential feature of the process of formation of calcium silicate hydrates by lime-clay reactions; this idea was seconded by Diamond, White, and Dolch (*32*) who envisioned the solution from the edges of the clay crystals as a possible mode of reaction. The addition of sodium hydroxide to a lime-soil mixture has been reported to accelerate strength development (*35, 13*). Moh (*36*) bases his explanation of these benefits in both soil-lime and soil-cement systems partly on the hypothesis of an increased rate of the solubility of potentially reactive silica.

The preceding arguments imply that the reaction path in lime-soil stabilization proceeds through solution—that silica liberated from the clay reacts with dissolved lime. In contrast, results of tests by the authors seem to suggest that reaction proceeds between adsorbed lime and the surface layers of the clay with which it is in contact. Such a mechanism has been proposed to account for the rapid generation of tobermorite gel by lime sorbed on silica gel surfaces at moderately elevated temperatures (*37*). Perhaps, as seems to be so in portland cement hydration, both *adsorbed-state* and *through-solution* reactions take place, both contributing to the development of the final product. However, a firm decision as to the reaction path awaits additional intensive experiments.

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(Continued on p. 273)

Experimental Isolation of Drivers' Visual Input

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

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Lack of scientific knowledge as to what features of the road and terrain drivers respond to prompted the study reported in this article. In an attempt to determine what these features might be, an experiment was conducted to measure drivers' visual inputs when their vision was isolated by a limiting aperture device. Use of this technique forced the driver to look for portions of the road from which he could get this essential information.

In the experiment, the shifts in fixation of the drivers' eyes emphasized the requirements of perceptual anticipation and vehicular alinement in which the driver must look far ahead to obtain a general idea of conditions that he will have to meet and at the same time keep his vehicle alined on the road by closeup viewing. Results obtained in the study indicate that the driver obtains this information from viewing the edges and centerline of the road. On this basis, edge markings are recommended where contrast is low or where night usage is heavy and a centerline is recommended, especially where two-way traffic is heavy.

Introduction

AN UNDERSTANDING of the driver's visual input provides a rational basis for many aspects of highway design. When the information that the driver uses is known, roads can be designed so that he will receive this information. As no experimental technique existed by which the visual factors in driving could be isolated, their nature primarily has been the subject of conjecture. From a scientific viewpoint, the isolation of visual input is both important and baffling. This information, used by the driver, sets off an action sequence of steering, braking, and acceleration of the car itself. Unless the stimulus that first triggers the driver's reactions is known, the consequent reactions are not easily understood. And when man's input has been specified, driving itself will be to a considerable extent described. The problem is difficult: The binocular field presents an enormous amount of information, but lack of communication lines to the driver's eyes or brain prevents a determination of how he selects and sorts this sensory input. If a driver is asked what he is responding to, suggestive, but in no sense trustworthy, answers are obtained.

A technique to determine what features of the road and terrain the driver is responding to is discussed in this article. The method involved having the driver guide the car while looking through a device containing a small aperture. By decreasing the visual field, the essential information, whatever it was, could not be seen at the same time: The driver was forced to obtain information in separate visual fixations. A continuous film record was made of the driver's field of view, and later it was analyzed to determine the center of his visual aim and the content of each fixation. The essential information he was using could be easily identified in each separate restricted fixation. This technique also may be used to determine the aircraft pilot's perceptual input and may be applicable, with modifications, to problems of human console design. Use of the aperture method was more advantageous than use of eye camera techniques, which provide a record of fixation position. The eye camera does not show the contribution of peripheral vision, and it does not provide a means for distinguishing essential from nonessential information.

Background

Different types of information have been suggested as being the inputs of driving (*1, 2, 3*).[3] A ranking of inputs made by a race driver and the editor of a scientific magazine, who evaluated their responses as drivers, to conspicuous features of the environment, may be taken as representative (*4*).

The race driver and the editor ranked the inputs to which they responded in the following order:

race driver		editor
View of the road ahead	1	View of the road ahead.
Seat-of-the-pants feel (transverse accelerations).	2	View of the car ahead.
Tachometer	3	Feel of steering wheel.
Feel of steering wheel	4	View of the road edge.
Engine sound	5	Speedometer.
View of the road behind	6	View of the road behind.
Oil-pressure gage	7	Seat-of-the-pants feel (transverse accelerations).
View of the car ahead	8	Engine sound.
View of the road edge	9	Blinking lights.
Smells	10	Tire noise.
Tire noise	11	Smells.
Blinking lights	12	Oil-pressure gage.
Speedometer	13	

The race driver and the scientific editor agreed that the view of the road ahead ranked first in affecting driver behavior. Otherwise, their ranking of environmental factors did not agree. This disagreement probably reflected a real difference in approach to driving. Obviously, a rigorous experimental method is required to determine what the driver is actually looking at and responding to. (Michaels and Cozan (*3*) have been perhaps the first to use rigorous experimental methods to validate a driving input. In field tests using lateral movement detectors, they showed that the driving response is inversely related to the sidewise drift of the approaching object or vehicle.)

Apparatus

An aperture device was developed that restricted the driver's vision and recorded his visual fixation positions. The main parts of this device are a head helmet, an aperture observation tube, and a camera. They are described in the following paragraphs and illustrated by figure 1.

A large plastic football helmet with frame supports was used to hold aperture and fiberglass pulpit. An inflatable bladder filled the space between helmet and head and held the helmet firmly in place.

An aperture observation tube 3½ inches in length and 1 inch in diameter was used. The tube could be raised or lowered to accommodate the observer's eye. Aperture disks of varying size could be fitted on the end of the tube. A circular screen covered the peripheral areas of the driver's field, and an eye patch fitted on his unused eye.

An 8-mm. camera was mounted coaxially with the aperture tube. The camera had an automatic (photoelectric) shutter and a battery-powered feed. The zoom lens was set at 8-mm. focal length. Speed of camera was slowed to 11 frames per second. The 25-foot film roll gave 178 seconds of record. This was enough to cover the experimental course.

Drivers

Ten volunteers (seven male, three female) from the Bureau of Public Roads served as test drivers. All subjects had vision rated sufficiently good to drive without glasses. The age of the drivers ranged from 19 to 38 years and the mean of their ages was 27.7. The years of driving experience of the volunteers ranged from 1 to 24 and the mean was 10.3.

Driving Course and Test Vehicle

A curved 2-lane road at the Herbert S. Fairbank Research Station, McLean, Va., where traffic density is low, was used as the test course. Figure 2 illustrates the test course: It was 22 feet wide, had a center yellow strip 4 inches wide, and was paved in blacktop. The shoulders of the road were planted in grass. The 2,805-foot length included a left

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[1] Presented at the 44th annual meeting of the Highway Research Board, Washington, D.C., January 1965.

[2] The advice and support of Dr. Richard M. Michaels, Science Advisor, Office of Research and Development, Bureau of Public Roads, is acknowledged by the author. Mildred Milazzo assisted in the preparation of this article by collecting and analyzing the data.

[3] References indicated by italic numbers in parentheses are listed on p. 273.

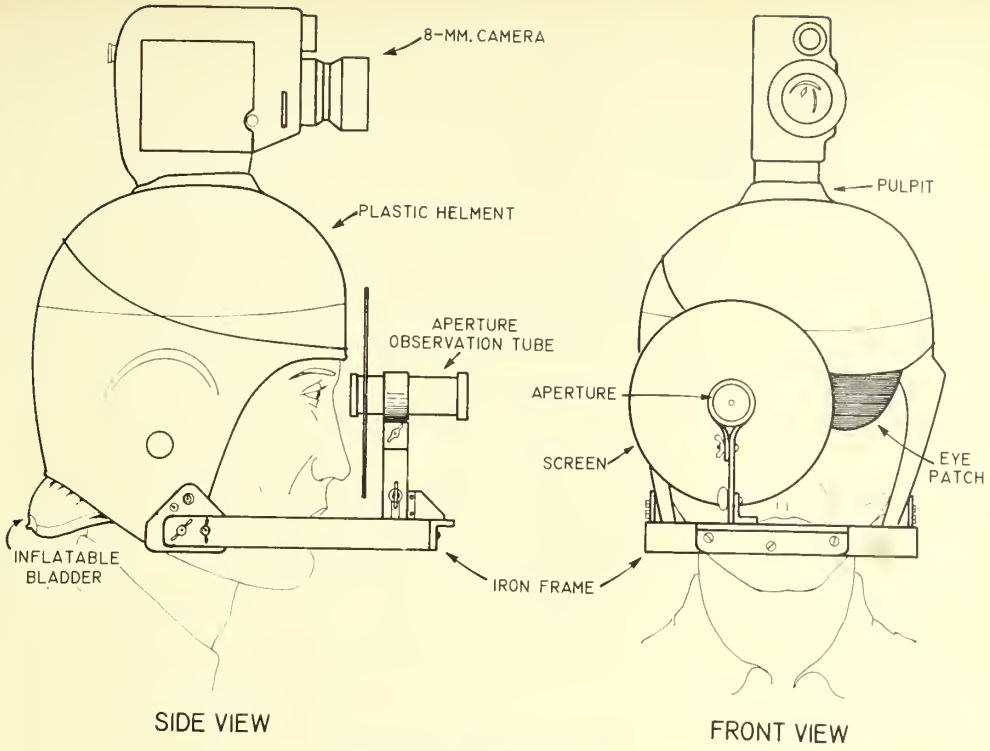

Figure 1.—Aperture device.

and a right curve. A carryall station wagon was used as the test vehicle. To obtain headroom for the helmet, the front seat of the vehicle was removed and a low cushion substituted for it. The test drivers could see over the hood without difficulty.

Procedure

The procedure included both practice and experimental phases. The program of these phases was carried out in a single 1-hour session, the procedure program is described in the following paragraphs.

The driver was instructed that he would be required to guide the car while his vision was restricted. He then practiced on a curved and hilly practice course, first without the aperture device and later with it. After practice, he was asked three questions:

- On the left curves, what did you look at mainly?
- On the right curves, what did you look at mainly?
- Was there any consistent pattern of movement of fixations from side to side or backward and forward that you adopted?

The driver operated in the experimental phase with: (1) open vision; (2) a large aperture of 9¾°; and (3) a small aperture of 4°. The open viewing was intended to provide a comparison of aperture driving with normal driving. The observer continually reported his visual fixation position on a tape recorder. After driving with open viewing, the drivers were asked the same three questions used in practice procedure.

Data collected during large aperture viewing were compared with data obtained during the small aperture viewing and an analysis made on the effect of size of viewing aperture. The procedure included calibration of the aperture device as well as data collection. The aperture device was calibrated for each driver by having the driver center in the aperture on a 2-inch square of white paper that was taped to a tree 50 feet away. Wooden uprights were adjusted until they appeared exactly on the left and right limits of the driver's field of view, as shown in figure 3. A brief film record of the adjustment was made for each driver and he was cautioned not to shake the helmet. The film was used as a record of the center and outer limits of each driver's field of view. As the driver drove the test course a continuous film record of his visual fixations and a tape recording of his verbal identifications of position were made. Camera and tape recordings were synchronized by the experimenter periodically interrupting the lens field and simultaneously recording an auditory signal. When the driver had completed the test run he was again asked the three questions.

The small aperture viewing was used to test the driver's visual behavior under the stress of limited information. The procedure used with the small aperture was essentially the same as that used with the larger aperture. Four drivers started at the A end of the track and four started at point B, as shown on figure 2. Two patterns of starting were used for each of the two groups of five drivers. The sequence of starting points was:

1st group	2d group
Open viewing—A, B, A	Open viewing—B, A, B
Large aperture—B, A, B	Large aperture—A, B, A
Small aperture—A, B, A	Small aperture—B, A, B

This variation in starting positions provided a replication of the procedure.

Analysis—Fixation Area and Fixation Position

Each driver's record was divided into separate fixations; that is, visual positions held until another was clearly assumed. Duration of each fixation was determined from the length shown on the film divided by known rate of film movement. A total of 3,305 separate fixations was analyzed on 4,152 inches of film recorded by the 10 drivers. The records were analyzed for fixation area and fixation position.

The area of fixation described the most inclusive road region covered in the small or large aperture. The sample areas are illustrated in figure 4. A response of the "whole road" area implied that both left and right edges of the road were included in the fixation. The "left lane" response included the left edge and centerline, and the "right lane" response covered the centerline and right edge. The designation for "yellow line," "left shoulder," and "right shoulder" applied to these features

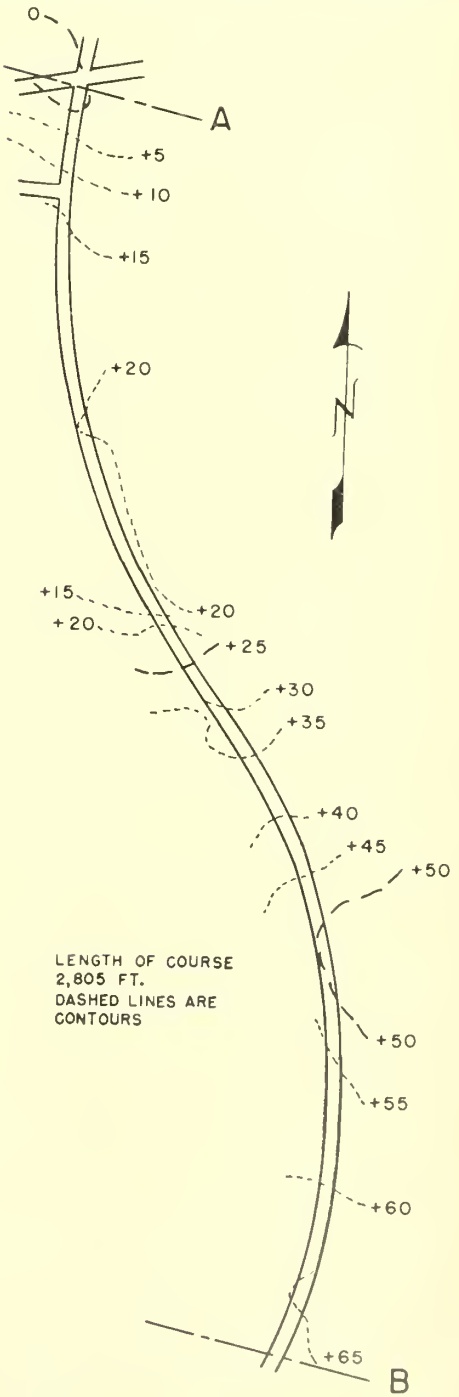

Figure 2.—Experimental course.

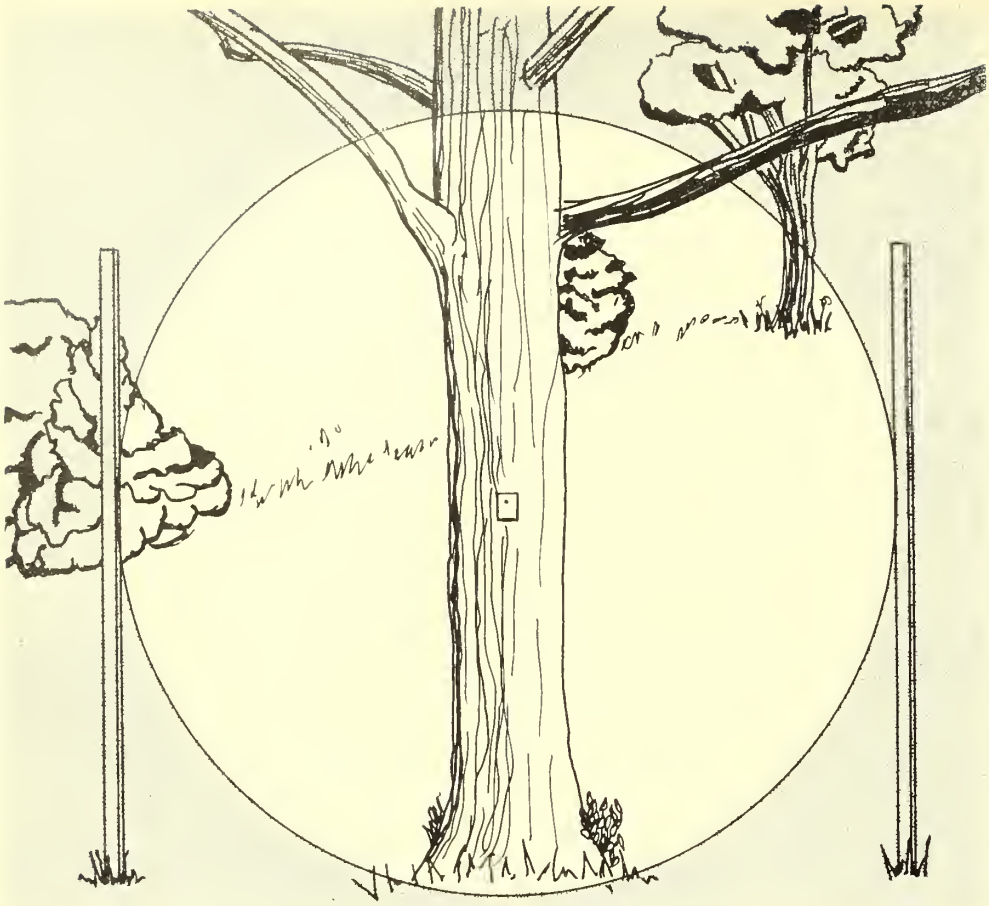

Figure 3.—Calibration arrangement.

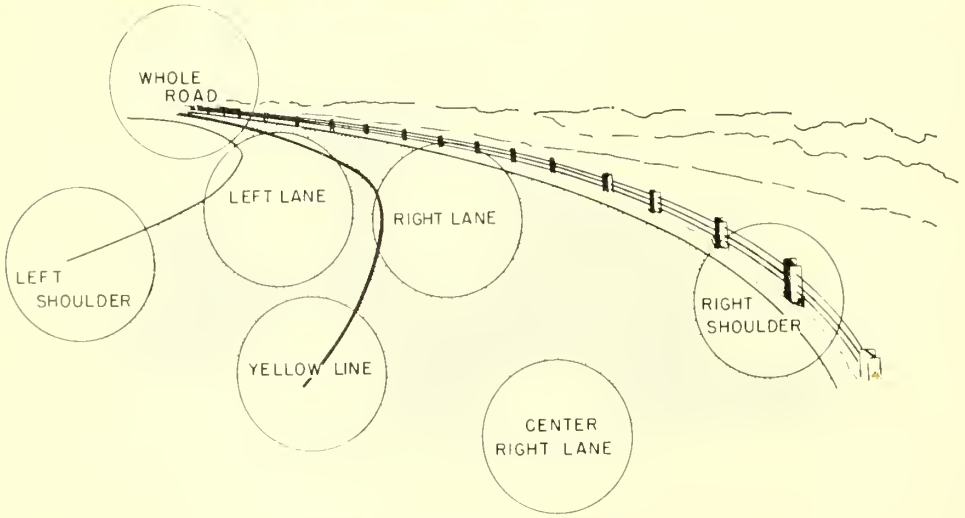

Figure 4.—Sample fixation areas.

singly. A few of the fixations were coded as "center of the left lane" or "center of the right lane." These areas did not include an edge or the centerline.

Fixation position designated the center of the fixation circle, which usually fell on the left shoulder, yellow line, right shoulder, or center of the right or left lane. Distance from the driver's eye was determined by noting on the film editor the width of the road at the center of aim. This width could then be compared with the precalibrated road widths at 50, 100, and 150 feet from the camera. The fixations could be classified as less than 50 feet, 50 to 100 feet, 100 to 150 feet, or more than 150 feet.

Essential Information Used by Driver

On the basis of the drivers' fixations, the road edges and the centerline are the essential information a driver needs. The importance of these features is shown by the summary results of fixation area and position given in tables 1 and 2. It is significant that 96.4 percent of small-aperture viewing and 99.8 percent of large-aperture viewing included an edge or center of the road. Fixation times coded under other areas may not have included an edge or the center.

The fixation position data given in table 2 confirm the importance of edge and centerline information; 80.9 percent of the large-aperture fixations and 85.7 percent of the small-aperture fixations were on edges or the centerline. Fixations not on these features included center of right lane, center of left lane, and trees on left and right. Considering the legitimate demands on a driver's attention of signs and passing cars, it is noteworthy that so high a proportion of fixations was centered on the road and edge borders.

The road edges and centerline also were referred to in the taped verbal statements made by the drivers while on the test route; these replies are summarized in table 3. Statements such as: "now my eye is on the yellow line," or "now my eye is on the right shoulder . . . near, etc." were coded as referring to the road. Such statements as "my vision just shifted back out where the sign is on the curve," or "there is an electric pole in front of us" were coded as not referring to the road. As illustrated, 96.3 percent of the comments made when the small aperture was used, 95.5 percent of the comments made when the large aperture was used, and 84.2 percent of the comments made when no aperture was used, referred to the road. The answers to the questions asked at the conclusion of each run support the conclusion that road edges and the centerline are the important fixation points for a driver. Each of the 10 drivers stated that he used the yellow line, and 9 of the 10 mentioned the right shoulder at least once during the trials. Their other comments referred mainly to anticipation and alinement, which are discussed later.

The finding that the driver depends upon road edges and the centerline for guidance has a number of implications for highway research. One is the desirability of marking the highway edges. Poor contrast of the road edge will affect the safety of the driver and the movement of traffic, particularly at night or in fog. These findings tend to support some of the theories on lateral guidance. One of the theories supported is that steering a car involves the maintenance of an acceptable, steady-state visual condition and the nulling of deviations from an acceptable state. The theory that the visual feedback for steering is slewing and sideslipping movements of road boundaries is also supported. However, the findings of the study reported here do not themselves prove these theories to be correct. The findings also imply the direction future research in this area might follow. Visibility studies should be undertaken of road edges and centerlines under night and fog conditions. The penalty of adverse visibility conditions and the advantages of using highway markings, reflectors, luminaires, and other devices should be assessed. The visibility of lane and edge markings can be assessed by using available formulas for luminance, size, and background contrast (*5*).

Table 1.—Time in seconds on different fixation areas—10 drivers, combined left and right curves

Distance, eye to fixation	Time of fixation in fixation areas											
	Left lane		Yellow line		Right lane		Whole road		Other areas		Total	
LARGE APERTURE												
Feet	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*
150 or more	51.82	4.30	2.58	0.21	140.84	11.70	486.42	40.38	------	------	681.66	56.59
100–150	26.58	2.20	3.08	0.26	53.41	4.43	156.77	13.02	------	------	239.84	19.91
50–100	12.98	1.08	4.41	0.37	63.02	5.23	155.07	12.87	1.00	0.08	236.48	19.63
Less than 50	3.45	0.29	5.60	0.46	12.15	1.01	24.56	2.04	0.82	0.07	46.58	3.87
Total	94.84	7.87	15.67	1.30	269.42	22.37	822.82	68.31	1.82	0.15	1,204.56	100.00
SMALL APERTURE												
150 or more	179.22	14.27	75.21	5.99	355.49	28.31	197.09	15.70	19.23	1.53	826.24	65.80
100–150	25.32	2.02	46.60	3.71	97.87	7.80	33.13	2.64	13.13	1.04	216.05	17.21
50–100	8.67	0.69	52.91	4.21	102.65	8.17	11.38	0.91	11.04	0.88	186.65	14.86
Less than 50	0.86	0.07	8.41	0.67	14.31	1.14	1.34	0.10	1.85	0.15	26.77	2.13
Total	214.07	17.05	183.13	14.58	570.32	45.42	242.94	19.35	45.25	3.60	1,255.71	100.00

Forward Reference Distance

Although all drivers guided the vehicle by reference to the road edges and the centerline, the manner in which information was obtained differed. The variation in approach is clearly illustrated in individual records given in tables 4–6. Driver G. P. tended to view the yellow centerline at a distance beyond 150 feet, thus including the yellow line or right lane in the field of view of the small aperture and the entire road in the large aperture. Driver G. W. looked mainly at the right shoulder. Driver R. O'C. shifted fixations between the centerline and the right shoulder. His main center of viewing was beyond 150 feet. These records refute the belief that a common sequence of viewing is shared by all drivers.

It has been proposed that the driver has a fixed forward fixation distance, which increases with vehicular speed (*6*). If the driver did not look ahead to compensate for man-vehicle reaction, he could not meet the current situation. Forward reference distance should increase with vehicular speed to compensate for increased stopping time.

To test the validity of the forward reference formulation, correlations were computed on the 10 test drivers between speed and average forward reference distance. Driver speed was obtained by determining the film time between known points on the course. Distance divided by time then indicated rate. The average forward reference distance was obtained by averaging fixation distances. Separate figures were obtained on the drivers who started at each of the two ends of the course. Correlations of forward reference position with speed, when the small aperture was used, were 0.55 and −0.37 and when the large aperture was used they were −0.52 and 0.15 (N=5). These results indicate no systematic relation between average forward reference distance and vehicular speed. The average fixation distance of 142 feet, which was the same for both apertures, seems to be longer than required to respond at average speeds of 13.4 m.p.h. for the small aperture or 14.7 m.p.h. for the large aperture. Even if a relation had existed between average forward reference distance and speed, it would not be very meaningful because of the variability of the fixation positions. The driver ordinarily looks far ahead of the car and then, seemingly in disregard of anticipation requirements, he may check his alinement with the road. This variability and the adjustment of fixation to particular road conditions makes the concept of average forward reference distance largely an abstraction.

Table 2.—Time in seconds on different fixation positions—10 drivers, combined left and right curves

Distance, eye to fixation	Time of fixation on fixation points											
	Left shoulder		Yellow line		Right shoulder		Center right lane		Center left lane		Trees left and right		Total	
LARGE APERTURE														
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	150.28	12.47	129.19	10.72	340.89	28.30	44.15	3.67	10.19	0.85	6.96	0.58	681.66	56.59
100–150	37.82	3.14	58.49	4.86	83.81	6.96	41.37	3.43	18.16	1.50	0.19	0.02	239.84	19.91
50–100	12.01	1.00	49.06	4.07	85.28	7.08	83.90	6.96	6.22	0.52	-----	----	236.48	19.63
Less than 50	-----	----	12.53	1.04	15.28	1.27	18.59	1.54	0.19	0.02	-----	----	46.58	3.87
Total	200.11	16.61	249.27	20.69	525.26	43.61	188.01	15.60	34.76	2.89	7.15	0.60	1,204.56	100.00
SMALL APERTURE														
150 or more	104.49	8.32	357.60	28.48	252.88	20.14	87.41	6.96	10.71	0.85	13.15	1.05	826.24	65.80
100–150	18.50	1.47	80.34	6.40	77.80	6.20	34.04	2.71	5.37	0.43	-----	----	216.05	17.21
50–100	2.72	0.22	79.91	6.36	79.11	6.30	21.65	1.72	3.26	0.26	-----	----	186.65	14.86
Less than 50	0.33	0.03	12.53	1.00	10.39	0.82	2.93	0.23	0.59	0.05	-----	----	26.77	2.13
Total	126.04	10.04	530.38	42.24	420.18	33.46	146.03	11.62	19.93	1.59	13.15	1.05	1,255.71	100.00

Table 3.—On-route identification of fixation positions—10 drivers

Test drivers	Statements identifying fixation positions					
	Open viewing		Large aperture		Small aperture	
	Number	References to road	Number	References to road	Number	References to road
E. C.	7	4	13	10	9	7
G. W.	28	26	15	14	11	11
D. U.	11	6	13	13	9	9
G. R.	23	22	34	34	38	38
L. H.	3	3	9	9	15	15
S. B.	33	33	65	65	58	58
G. P.	16	14	11	11	22	20
R. O'C.	37	32	60	59	105	103
B. C.	18	14	30	24	30	27
F. T.	11	9	15	14	25	22
Total	187	163	265	253	322	310
References to road (pct.)	87.2		95.5		96.3	

Left and Right Curves

The fixation pattern differed somewhat on left and right curves, as shown in tables 7, 8, 9, and 10. The point of fixation in distant vision deserves special attention. Signs and other visual aids should presumably be located where the motorists eye tends to fall. When drivers looked through the large aperture, their eye fell 6.48 feet from the right edge of the road (left curve average of fixations beyond 150 feet). The corresponding figure for the right curve was 8.91 feet. When the small aperture was used, the drivers' eyes fell 6.95 feet from the right edge on a left curve and 10.38 feet on a right curve. The average shifts in fixation position are statistically significant at the 0.01 level (*t*-test). Apparently the eye moves to the left on a right curve; but on the average, it does not cross into the opposing lane. The results support the practice of placement of signs on the right side of the road.

Perceptual Anticipation and Alinement

Most records show continuous visual shifts, forward to the limit of the road and backward toward the vehicle. The record of driver E. C. shown in figure 5 illustrates these movements. On the left curve, rapid fixation movements occurred between positions beyond 150 feet and those less than 50 feet from the driver. These shifts also occurred along the right shoulder when the driver was on a right curve. When the small aperture was used, 48 percent of all drivers' fixation sequences crossed the zone borders at 50, 100, or 150 feet, and when the large aperture was used, 55 percent of the drivers' sequences crossed the zone borders at these distances. These movements may be explained by the con-

Table 4, Part 1.—Time in seconds required by driver G. P. on different fixation areas and positions through small aperture—left curve

Distance, eye to fixation	Time required for fixation area												Time required for fixation position							
	Left lane		Yellow line		Right lane		Whole road		Left shoulder		Right shoulder		Total		Left shoulder		Yellow line		Right shoulder		Total	
	LEFT CURVE, SMALL APERTURE														LEFT CURVE, SMALL APERTURE							
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	35.47	23.9	37.73	25.5	18.61	12.6	38.26	25.8	7.12	4.8	2.63	1.8	139.82	94.4	3.26	2.2	124.11	83.8	12.45	8.4	139.82	94.4
100–150	---	---	4.67	3.1	---	---	---	---	---	---	---	---	4.67	3.1	---	---	4.67	3.1	---	---	4.67	3.1
50–100	---	---	2.45	1.6	---	---	0.70	0.5	---	---	0.11	0.1	---	---	---	---	3.15	2.1	0.11	0.1	3.26	2.2
Less than 50	---	---	0.41	0.3	---	---	---	---	---	---	---	---	0.41	0.3	---	---	0.41	0.3	---	---	0.41	0.3
Total	35.47	23.9	45.26	30.5	18.61	12.6	38.96	26.3	.12	4.8	2.74	1.9	148.16	100.0	3.26	2.2	132.34	89.3	12.56	8.5	148.16	100.0

Table 4, Part 2.—Time in seconds required by driver G. P. on different fixation areas and positions through large aperture—right curve

Distance, eye to fixation	Time required for fixation area						Time required for fixation position									
	Yellow line		Whole road		Total		Left shoulder		Yellow line		Right shoulder		Trees		Total	
	RIGHT CURVE, LARGE APERTURE						RIGHT CURVE, LARGE APERTURE									
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	0.82	0.7	111.43	93.3	112.25	94.0	27.69	23.2	39.60	33.2	43.74	36.6	1.22	1.0	112.25	94.0
100–150	0.89	0.8	4.67	3.9	5.56	4.7	0.74	0.6	3.22	2.7	1.60	1.4	-----	-----	5.56	4.7
50–100	-----	-----	1.19	1.0	1.19	1.0	-----	-----	1.19	1.0	-----	-----	-----	-----	1.19	1.0
Less than 50	-----	-----	0.37	0.3	0.37	0.3	-----	-----	0.37	0.3	-----	-----	-----	-----	0.37	0.3
Total	1.71	1.5	117.66	98.5	119.37	100.0	28.43	23.8	44.38	37.2	45.34	38.0	1.22	1.0	119.37	100.0

tradictory requirements of perceptual anticipation and vehicular alinement. Perceptual anticipation requires the driver to look far ahead to obtain a general idea of conditions that will have to be met. Alinement behavior requires closeup viewing to ensure that the vehicle is on the road.

The drivers mentioned anticipation and alinement in explaining how they guided the car. Excerpts from the record illustrate these activities:

Question: How did you guide yourself on the right curve.

Table 5.—Time in seconds used by driver G. W. on different fixation areas and positions—combined left and right curves

Distance, eye to fixation	Time of fixation															
	Fixation area						Fixation position									
	Right lane		Whole road		Total		Left shoulder		Yellow line		Right shoulder		Center right lane		Total	
	SMALL APERTURE															
	Seconds	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*	*Seconds*	*Percent*
150 or more	8.23	8.4	7.12	7.2	15.35	15.6	0.41	0.4	2.74	2.8	12.20	12.4	-----	-----	15.35	15.6
100–150	30.40	30.8	3.15	3.2	33.55	34.0	-----	-----	0.26	0.2	32.92	33.4	0.37	0.4	33.55	34.0
50–100	47.15	47.9	0.11	0.1	47.26	48.0	-----	-----	0.11	0.1	47.15	47.9	-----	-----	47.26	48.0
Less than 50	2.37	2.4	-----	-----	2.37	2.4	-----	-----	-----	-----	2.37	2.4	-----	-----	2.37	2.4
Total	88.15	89.5	10.38	10.5	98.53	100.0	0.41	0.4	3.11	3.1	94.64	96.1	0.37	0.4	98.53	100.0
	LARGE APERTURE															
150 or more	32.66	30.8	64.20	60.7	96.86	91.5	13.72	13.0	6.86	6.5	72.65	68.6	3.63	3.4	96.86	91.5
100–150	7.34	7.0	0.56	0.5	7.90	7.5	-----	-----	-----	-----	7.90	7.5	-----	-----	7.90	7.5
50–100	0.52	0.5	0.52	0.5	1.04	1.0	-----	-----	0.52	0.5	0.52	0.5	-----	-----	1.04	1.0
Less than 50	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Total	40.52	38.3	65.28	61.7	105.80	100.0	13.72	13.0	7.38	7.0	81.07	76.6	3.63	3.4	105.80	100.0

Table 6.—Time in seconds used by driver R. O'C. on different fixation areas and positions—combined left and right curves

Distance, eye to fixation	Time of fixation																					
	Fixation area										Fixation position											
	Left lane		Yellow line		Right lane		Whole road		Total		Left shoulder		Yellow line		Right shoulder		Center right lane		Center left lane		Total	
	SMALL APERTURE																					
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	53.45	31.5	10.94	6.4	56.64	33.3	31.17	18.4	152.20	89.6	22.28	13.1	57.90	34.1	42.00	24.7	28.20	16.6	1.82	1.1	152.20	89.6
100–150	1.74	1.0	2.19	1.3	13.16	7.7	-----	----	17.09	10.0	0.07	0.0	4.82	2.8	9.68	5.7	2.52	1.5	----	----	17.09	10.0
50–100	----	----	-----	----	0.63	0.4	-----	----	0.63	0.4	-----	----	-----	----	0.63	0.4	-----	----	----	----	0.63	0.4
Less than 50	-----	----	-----	----	-----	----	-----	----	-----	-----	-----	----	-----	----	-----	----	-----	----	----	----	-----	-----
Total	55.19	32.5	13.13	7.7	70.43	41.4	31.17	18.4	169.92	100.0	22.35	13.1	62.72	36.9	52.31	30.8	30.72	18.1	1.82	1.1	169.92	100.0
	LARGE APERTURE																					
150 or more	NA	NA	NA	NA	20.80	16.2	79.39	56.1	93.19	72.3	18.61	14.4	12.49	9.7	60.20	46.7	1.89	1.5	NA	NA	93.19	72.3
100–150	NA	NA	NA	NA	7.12	5.5	27.58	21.4	34.70	26.9	4.97	3.9	5.56	4.3	21.09	16.3	3.08	2.4	NA	NA	34.70	26.9
50–100	NA	NA	NA	NA	1.07	0.8	-----	----	1.07	9.8	-----	----	-----	----	0.18	0.1	0.89	0.7	NA	NA	1.07	0.8
Less than 50	NA	NA	NA	NA	-----	----	-----	----	-----	-----	-----	----	-----	----	-----	----	-----	----	NA	NA	-----	-----
Total	NA	NA	NA	NA	28.99	22.5	99.97	77.5	127.96	100.0	23.58	18.3	18.05	14.0	81.47	63.1	5.86	4.6	NA	NA	128.96	100.0

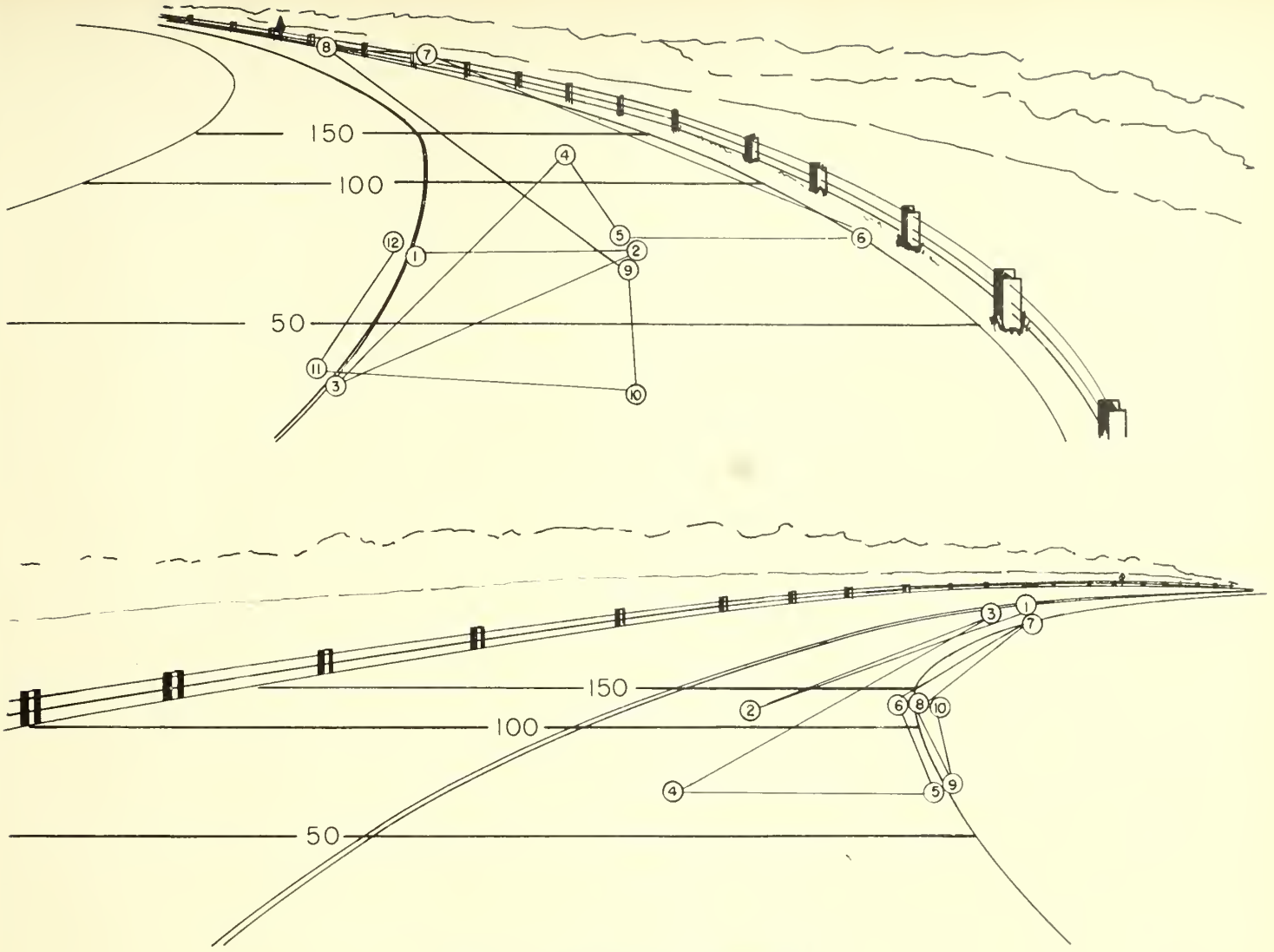

Figure 5.—Anticipation and alinement.

Answer by test driver G. P. when small aperture was used: "Well, I just saw the same thing. My vision would go out to the curve. I'd see as much as I could and then come back again. One time I stayed out too long and I was out of the road."

Question: Was there any pattern of movement.

Answer by test driver R. O'C. when small aperture was used: "Just that I . . . you look ahead frequently to see the whole situation—then come back to your immediate points of reference. I wanted to see what was ahead and then put myself within the lane by something closer—centerline or shoulder line."

Question: How did you guide yourself on the left curve.

Answer by test driver N. M. when no aperture limitations were imposed: "Going into a left turn, generally I was looking at the center strip and the curvature in the distance along the left side of the road. And as I approached the curve—got into the curve—I was looking generally at the center strip and the right side of the shoulder."

The author recommends that the driver should always be given an unimpeded view ahead that is sufficient to satisfy his anticipation requirements. The fact that drivers need to perceptually anticipate conditions ahead has application to road design. This need to anticipate conditions ahead has been recognized in traffic regulation and guidance, road design manuals, and comes out clearly in the data collected in the study reported here. Little is known about perceptual anticipation requirements in the variety of situations met on the highway and further research in this area may be very profitable.

Methodological Problems

Two of the methodological problems encountered in the study discussed here are: (1) The adequacy of introspective data (on-route

Table 7.—Summary of time required by 10 drivers, using aperture, on different fixation positions

Distance, eye to fixation	Time required for fixation													
	Left shoulder		Yellow line		Right shoulder		Center right lane		Center left lane		Trees, left and right sides		Total	
						LEFT CURVE								
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	49.86	7.91	88.34	14.02	192.06	30.50	19.05	3.03	8.63	1.37	6.59	1.04	364.53	57.87
100–150	14.68	2.33	37.66	5.98	47.89	7.60	25.02	3.97	17.05	2.71	0.19	0.03	142.49	22.62
50–100	6.79	1.08	35.45	5.63	15.89	2.52	42.22	6.70	4.52	0.72	---	----	104.87	16.65
Less than 50	-----	----	9.82	1.56	1.45	0.23	6.56	1.04	0.19	0.03	----	----	18.02	2.86
Total	71.33	11.32	171.27	27.19	257.29	40.85	92.85	14.74	30.39	4.83	6.78	1.07	629.91	100.00
						RIGHT CURVE								
150 or more	100.42	17.47	40.85	7.11	148.83	25.90	25.10	4.37	1.56	0.27	0.37	0.07	317.13	55.19
100–150	23.14	4.03	20.83	3.62	35.92	6.62	16.35	2.85	1.11	0.19	----	- --	97.35	16.94
50–100	5.22	0.91	13.61	2.37	69.39	12.07	41.68	7.25	1.70	0.30	----	----	131.60	22.90
Less than 50	-----	----	2.71	0.47	13.83	2.41	12.03	2.09	----	----	----	----	28.57	4.97
Total	128.78	22.41	78.00	13.57	267.97	46.63	95.16	16.56	4.37	0.76	0.37	0.07	574.65	100.00

statements of fixation, answers to questionnaires) for indicating the driver's visual input, and (2) the effects of the stress imposed by limiting information with the small aperture. The on-route statements and the questionnaire data were suggestive, but the introspective data had a shortcoming of incompleteness. The 3,305 separate film fixations obtained permitted a reliable analysis of the distribution and sequence of eye positionings. In contrast, the 563 statements given on-route and the 20 questionnaire responses offered only a skeletal indication of what was going on. However, the introspective data provided confirming evidence of the validity of the film analysis, and they were valuable in revealing the drivers' purposes in perceptual anticipation and alinement. These uses and limitations of introspective data are similar to those generally encountered in experimental work. The complexity of fixation data has been noted in previous eye camera studies (7).

The adaptation made by drivers to the stress of limited information is shown by a comparison of results from tests in which the small and large apertures were used (tables 1 and 2). When looking through the small aperture, drivers fixated the yellow line 42.4 percent of the time and the right shoulder 33.5 percent, and this enabled them to track the yellow line or view the entire right lane. When looking through the large aperture, drivers could see the entire road and this inclusive feature was used 68.3 percent of the time; the right lane was used 22.4 percent of the time as it could be covered even when the driver looked close to the car. The records seem to indicate that the driver selects a view that permits him to do the job. Under stress, this may be as simple as holding fast to the yellow line and tracking it, as illustrated by the record of G. P. given in table 4. However, the driver prefers to see more of the road; even when the small aperture was used, the entire right lane was viewed 45.4 percent of the time and the entire road 19.4 percent of the time.

Recommendations

The experimental evidence of the study reported here is the basis of suggestions for the design of highway markings. The experimental evidence must be weighed along with considerations of cost, public acceptance, current usage, and similar factors before it can be applied. But experimental findings have special status because they are perhaps less subject to debate than other considerations.

Results of the study provide a rationalization for edge and center lane markings. The question remains as to which features should be emphasized by being specially marked. The evidence indicates that driving can be done by use of only the centerline (table 1, small aperture). This might lead to the recommendation that only the centerline be presented. However, drivers prefer to see the right lane and the entire road, as evidenced by the large proportion of total time spent on these features. Factors other than those considered in the experiment reported in this article will dictate road marking policy. For example, road edges differ in luminance and contrast, and hence the need for them to be painted. Where contrast is low or where night usage is heavy, edge markings are recommended. Where two-way traffic is heavy, a centerline should be used.

Table 8.—Summary of time required by 10 drivers, using large aperture, on different fixation areas

Distance, eye to fixation	Time required for fixation											
	Left lane		Yellow line		Right lane		Whole road		Other areas [1]		Total	
LEFT CURVE												
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	5.41	0.85	2.58	0.41	135.72	21.55	220.82	35.06	----	----	364.53	57.87
100–150	6.52	1.04	3.08	0.49	53.41	8.48	79.48	12.61	----	----	142.49	22.62
50–100	3.60	0.57	4.41	0.70	52.42	8.32	43.44	6.90	1.00	0.16	104.87	16.65
Less than 50	1.19	0.19	5.23	0.83	6.67	1.06	4.11	0.65	0.82	0.13	18.02	2.86
Total	16.72	2.65	15.30	2.43	248.22	39.41	347.85	55.22	1.82	0.29	629.91	100.00
RIGHT CURVE												
150 or more	46.41	8.08	----	----	5.12	0.89	265.60	46.22	NA	NA	317.13	55.19
100–150	20.06	3.49	----	----	----	----	77.29	13.45	NA	NA	97.35	16.94
50–100	9.38	1.63	----	----	10.60	1.85	111.63	19.42	NA	NA	131.60	22.90
Less than 50	2.26	0.39	0.37	0.07	5.48	0.95	20.45	3.56	NA	NA	28.57	4.97
Total	78.11	13.59	0.37	0.07	21.20	3.69	474.97	82.65	NA	NA	574.65	100.00

[1] Left shoulder, center left lane, center right lane, right shoulder.

Table 9.—Summary of time in seconds required by 10 drivers, using small aperture, on different fixation positions

Distance, eye to fixation	Time required for fixation													
	Left shoulder		Yellow line		Right shoulder		Center right lane		Center left lane		Trees, left and right		Total	
LEFT CURVE														
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	33.40	4.50	174.56	23.50	176.97	23.83	56.08	7.55	10.71	1.44	11.20	1.51	462.92	62.33
100–150	11.23	1.51	53.56	7.21	35.55	4.79	14.86	2.00	5.38	0.73	----	----	120.58	16.24
50–100	2.37	0.32	64.17	8.64	58.05	7.81	7.71	1.03	3.00	0.40	----	----	135.30	18.20
Less than 50	0.33	0.04	12.34	1.66	8.86	1.19	1.90	0.26	0.59	0.08	----	----	24.02	3.23
Total	47.33	6.37	304.63	41.01	279.43	37.62	80.55	10.84	19.68	2.65	11.20	1.51	742.82	100.00
RIGHT CURVE														
150 or more	71.10	13.87	183.05	35.69	75.92	14.80	31.32	6.11	----	----	1.96	0.38	363.35	70.85
100–150	7.27	1.42	26.76	5.22	42.26	8.24	19.16	3.73	----	----	----	----	95.45	18.61
50–100	0.33	0.06	15.75	3.07	21.06	4.11	13.94	2.72	0.26	0.05	----	----	51.34	10.01
Less than 50	-----	-----	0.19	0.04	1.51	0.29	1.04	0.20	----	----	----	----	2.74	0.53
Total	78.70	15.35	225.75	44.02	140.75	27.44	65.46	12.76	0.26	0.05	1.96	0.38	512.88	100.00

Table 10.—Summary of time in seconds required by 10 drivers, using small aperture, on different fixation areas

Distance, eye to fixation	Time required for fixation											
	Yellow line		Left lane		Whole road		Right lane		Other areas [1]		Total	
LEFT CURVE												
Feet	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*	*Sec.*	*Pct.*
150 or more	53.34	7.18	21.53	2.90	68.84	9.27	310.69	41.83	8.52	1.15	462.92	62.33
100–150	37.03	5.00	6.64	0.89	12.93	1.74	56.00	7.54	7.98	1.07	120.58	16.24
50–100	43.19	5.80	2.70	0.36	6.82	0.92	77.59	10.45	5.00	0.67	135.30	18.20
Less than 50	8.41	1.13	0.86	0.12	1.15	0.15	12.79	1.72	0.81	0.11	24.02	3.23
Total	141.97	19.11	31.73	4.27	89.74	12.08	457.07	61.54	22.31	3.00	742.82	100.00
RIGHT CURVE												
150 or more	21.87	4.26	157.70	30.75	128.25	25.01	44.82	8.74	10.71	2.09	363.35	70.85
100–150	9.57	1.87	18.68	3.64	20.20	3.94	41.85	8.16	5.15	1.00	95.45	18.61
50–100	9.72	1.90	5.96	1.16	4.56	0.88	25.06	4.89	6.04	1.18	51.34	10.01
Less than 50	------	------	------	------	0.18	0.04	1.52	0.29	1.04	0.20	2.74	0.53
Total	41.16	8.03	182.34	35.55	153.19	29.87	113.25	22.08	22.94	4.47	512.88	100.00

[1] Right shoulders, left shoulders, center right lane, center left lane.

The analysis of fixations on curves indicates that a driver tends to shift his distance fixation to the left on a right curve. However, the movement is not large enough to move the average fixation position across into the opposing lane. Presumably then, highway markings should be presented on the right side of the road.

Summary and Conclusions

A technique to determine what features of the road and terrain the driver is responding to is presented in this article. The method involved having the driver guide the car while looking through a device containing a small aperture. By decreasing the visual field, the essential information, whatever it is, cannot be seen at once: the driver is forced to obtain this information in separate visual fixations. A continuous film record was made of the driver's field of view and later analyzed to indicate the center of his visual aim and the content of each fixation. This aperture device was used to obtain visual positional data on 10 drivers who followed a 2-lane road, where traffic density was low. The film records provided 3,305 separate fixations, which were coded for position, distance from the eye, and duration. Conclusions made on the basis of the analysis were:

- The essential information required by a driver is provided by the road edges and center lane marker. At least one of these road features was included in 98.2 percent of the fixations made when the large aperture was used. The drivers' on-route statements of fixation position, and the answers to a questionnaire also indicated the importance of the road edges and lane marker.
- Although all drivers utilize the road edges and centerline to guide the vehicle, the manner in which this information is obtained differs from subject to subject. The film records refuted the idea that the driver has a fixed point of forward reference or that a common pattern of viewing is shared by all drivers.
- In going from a left to a right curve, the position of fixation tends to shift in the opposite direction, that is from right to left. However, the average point of fixation beyond 150 feet did not cross the centerline into the opposing lane.
- The persistent pattern of fixation movements, forward to the limits of the road and back again to the vehicle, are explained by the contradictory requirements of perceptual anticipation and vehicular alinement with the road. This conclusion and the preceding one should be tested further, if possible, with eye-camera techniques.

Methodological problems concerning the adequacy of introspective data for determining the driver's visual input, and the stress of small-aperture viewing have been discussed. The implications of results to the placement of signs and highway markings have been presented.

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MECHANISMS OF SOIL-LIME STABILIZATION

(Continued from p. 265)

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Travel Habits in Cities of 100,000 or More

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

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The preferences for certain modes of transportation by the urban population are considered in the survey reported in this article. These preferences are expected to be helpful in planning for highways and transportation systems to meet the needs of an expected urban population of 230 million by 1990.

Travel habits, income, and other information about the economics of persons living in cities of 100,000 or more population are discussed. Information is included on the availability of the nearest public transportation to work and the mode of travel to work: the relationship between family income and the mode of travel to work; and the distribution of automobile trips and travel to the downtown shopping areas, by purpose and length of trip.

During the spring of 1961, a nationwide automobile-use survey was made for the Bureau of Public Roads by the Bureau of the Census. In addition to the characteristics of automobile use by households, data were collected on income and composition of families for different population groupings. Because it has been estimated that 80 percent of the people in the United States will be living in urban areas by 1990, the preferences for certain modes of transportation and of how these preferences may influence the direction of future transportation development must be considered in current planning.

Introduction

THE PURPOSE of this article is to present some information on the travel characteristics of workers now living in cities, exclusive of the surrounding urban areas, where the population is 100,000 or more. The data considered are the choice of travel modes used by workers related to the nearness of public transportation to work, the distance to work, and the family income. Also considered is the distribution of trips and travel to the downtown area by purpose of trips and, for each purpose, the proportion of all trips destined for the downtown area.

The principal data on which this article is based were derived from a nationwide automobile-use survey conducted in the spring of 1961 by the Bureau of the Census under contract to the Bureau of Public Roads, supplemented by information that since has become available. The sample used by the Bureau of the Census was approximately 5,000 dwelling units from the Current Population Survey. This survey, conducted monthly by the Bureau of the Census, is based on a statistically selected sample representing the noninstitutional civilian population. It is made to obtain current information on employment, unemployment, and related economics. Data from more than 1,300 of the 5,000 households in the sample were collected for the study reported in this article. These households were located in cities having a population of 100,000 or more. Because these data were based on a probability sampling of households, the information is subject to sampling variability.

The term "sampling variability" refers to the differences that might be expected between results of sample surveys and the results that would have been obtained from a complete enumeration of all households. Based on the estimates of sampling variability that occurred as a result of the use of the sample of 1,300, the odds were about 2 to 1 that the estimated number of trips would differ by no more than 10 percent from the unknown value that would have been obtained had the complete enumeration been made. The comparable measure of relative sampling variability for the estimate of vehicle miles was 20 percent for the same odds.

Conclusions

Information related to travel characteristics of residents of large cities, obtained from sources that are valid for nationwide (but not local) comparisons, has been presented in this article. From this information, it is concluded that the automobile is the mode of transportation used by a large proportion of the residents of large cities, regardless of the availability of other modes. Because a worker lives close to public transportation does not necessarily influence his use of it as his regular means of getting to and from work. Neither does closeness of home to work by itself prompt a worker to forego use of his automobile for trips to work. Use of the automobile for trips to work tends to increase as income increases.

Although only one-seventh of all automobile trips made for purposes of earning a living were destined for the downtown area, more than half of all the trips having downtown destinations were made for this purpose.

Background

An assumption can be made that the major portion of highway needs of this country will be concentrated in urban areas in the next two or three decades. Probably by 1990 about 80 percent of the total population of the 48 contiguous States and the District of Columbia will be residing in urban areas. Projections of future population range from 262 million to 301 million for the year 1990. A recent projection prepared by the Bureau of Public Roads for 1990 is 286 million. If the probability holds that 80 percent would be residents of urban areas, the resultant urban population would be 230 million.

The planning and construeting of the urban highway system—or, for that matter, systems including all modes of urban transportation—to meet the travel needs and desires of 230 million persons is a task of unparalleled magnitude in transportation history. As more and more people live in urban concentrations, planning for transportation systems becomes increasingly necessary. Such systems should embrace the different modes of transportation in a balance that will provide efficient and effective service for States and local communities. The study of travel habits of workers reported here provides some information on preferences for certain modes of transportation and how these preferences may suggest the trend of future transportation development.

Transportation to Work

Some form of public transportation was available within 2 blocks of the homes of 69.3 percent of all workers who lived in places having a population of 100,000 or more, as shown in table 1. At the other extreme, 8 percent had no public transportation available for trips to work. Public transportation was available 6 blocks or less from home for 87 percent of all workers. Furthermore, of the workers living 15 miles or more from work, 15.6 percent had no public transportation available nearer than 6 blocks or less from home; for workers living closer than 15 miles to work, less than 5 percent had no public transportation closer than 6 blocks from their homes.

Half of all workers who had some form of public transportation available for trips to work chose to use automobiles for this purpose, as shown in table 2. The percentage was a little more than the average for workers who lived within 2 blocks of public transportation. Possible reasons for this might have been inadequacy of or dissatisfaction with the public transportation system.

The time factor may have entered into the choice of the automobile as the means of getting to work. The Bureau of the Census has compiled data in the Preliminary Progress Report, Home to Work Travel Survey, 1963 Census of Transportation. In the data shown

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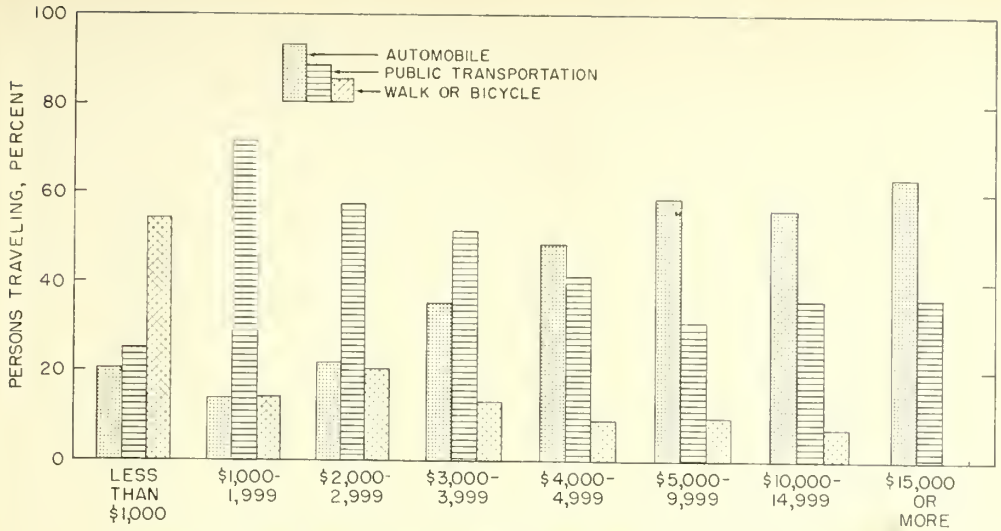

Figure 1.—Distribution of persons in each family group, by mode of home-to-work transportation.

for central cities of standard metropolitan statistical areas, 74 percent of the persons commuting to work by automobile required less than 25 minutes to get there, whereas only 25 percent of the workers commuting by public transportation were able to get to work within that time. These figures may be an indication of the importance that commuters attach to time savings.

Of the one-half of all workers who did not choose to go to work by automobile, 38.6 percent used public transportation and 11.5 percent walked or used a bicycle, as shown in table 3. Where the distance to work was less than 1 mile, 74.9 percent of the workers either walked or rode a bicycle. But even at this relatively short distance from home to work, 14 percent chose to go by automobile and 11.1 percent used public transportation. Moreover, most of those who went by automobile were drivers, not passengers.

The mode of travel to work, by 1-mile increments from 1 to 4.9 miles and then by 5-mile increments, is shown in table 3. Generally, the longer the distance from work, the larger the proportion was of workers using automobiles for home-to-work transportation. Data in the table also indicate that mileage to work influences the extent of car pooling. As the distance to work increased to more than 3 miles, a higher proportion of workers were reported as automobile passengers, the range being from 5.4 percent in the 4- to 4.9-mileage group to 12.7 percent in the 20- to 24.9-mileage group.

Family Income and Mode of Travel

The choice of mode of travel to work is undoubtedly influenced by many factors, singly or in combination. Probably, at least two and possibly more of these factors enter

Table 1.—Percentage distribution of workers, by distance to nearest public transportation to work [1]

Distance to work, one way	Nearest public transportation, by workers			
	1–2 blocks	3–6 blocks	More than 6 blocks	None available
Miles	*Percent*	*Percent*	*Percent*	*Percent*
Less than 5	71.7	15.5	2.8	10.0
5.0–9.9	69.0	23.0	4.7	3.3
10.0–14.9	71.5	19.5	3.0	6.0
15.0 or more	56.2	13.9	15.6	14.3
All distances	69.3	18.0	4.7	8.0

[1] Nationwide automobile-use survey, spring 1961—for persons living in places having a population of 100,000 or more.

Table 2.—Percentage distribution of workers using automobiles for trips to work, by distance to nearest public transportation to work [1]

Distance to work, one way	Nearest public transportation, for workers [2]			
	1–2 blocks	3–6 blocks	More than 6 blocks	All distances
Miles	*Percent*	*Percent*	*Percent*	*Percent*
Less than 5	44.8	46.1	56.7	45.4
5.0–9.9	61.2	41.1	39.7	55.3
10.0–14.9	54.3	60.3	51.4	55.4
15.0 or more	53.6	60.3	31.6	50.7
All distances	51.5	47.3	42.8	50.2

[1] Nationwide automobile-use survey, spring 1961—persons living in places having a population of 100,000 or more.
[2] Excludes persons for whom no public transportation was available.

Table 3.—Percentage distribution of workers, by mode of travel and distance to work [1]

Distance to work, one way	Mode of travel, by workers				
	Automobile			Public transportation or combination [2]	Walk or bicycle
	Driver	Passenger	Total		
Miles	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Less than 1	13.1	0.9	14.0	11.1	74.9
1.0–1.9	40.2	4.8	45.0	30.0	25.0
2.0–2.9	41.3	2.1	43.4	51.8	4.8
3.0–3.9	51.0	8.6	59.6	38.7	1.7
4.0–4.9	55.5	5.4	60.9	39.1	[3]
5.0–9.9	48.6	8.5	57.1	42.3	0.6
10.0–14.9	50.5	7.2	57.7	42.3	[3]
15.0–19.9	41.3	8.1	49.4	50.6	[3]
20.0–24.9	52.8	12.7	65.5	34.5	[3]
25.0 or more	55.4	8.6	64.0	36.0	[3]
Unknown [4]	19.7	19.7	39.4	57.4	3.2
Total	43.2	6.7	49.9	38.6	11.5

[1] Nationwide automobile-use study, spring 1961—persons living in places having a population of 100,000 or more.
[2] Public transportation or public transportation and automobile.
[3] Less than 0.1.
[4] Distance not reported, less than 3 percent of that reported.

Table 4.—Percentage distribution of workers, by family income and mode of home-to-work travel [1]

Family income	Mode of travel, by workers				
	Automobile			Public transportation or combination [2]	Walk or bicycle
	Driver	Passenger	Total		
Dollars	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Less than 1,000	15.9	4.7	20.6	25.1	54.3
1,000–1,999	14.1	[3]	14.1	71.7	14.2
2,000–2,999	18.9	3.0	21.9	57.5	20.6
3,000–3,999	26.7	8.5	35.2	51.5	13.3
4,000–4,999	36.6	12.0	48.6	41.4	9.0
5,000–9,999	50.0	9.1	59.1	31.2	9.7
10,000–14,999	54.3	1.9	56.2	36.6	7.2
15,000 or more	63.3	[3]	63.3	36.7	[3]
Unknown [4]	41.7	4.2	45.9	39.8	14.3
Total	43.2	6.7	49.9	38.6	11.5

[1] Nationwide automobile-use study, spring 1961—persons living in places having a population of 100,000 or more.
[2] Public transportation or public transportation and automobile.
[3] Less than 0.1.
[4] Income not reported, 13 percent of sample.

Table 5.—Percentage distribution of automobile trips, by purpose and family income [1]

Purpose of trip	Percentage of trips for families having incomes of—								
	Less than $1,000	$1,000–1,999	$2,000–2,999	$3,000–3,999	$4,000–4,999	$5,000–9,999	$10,000–14,999	$15,000 or more	All income
Earning a living:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
To and from work	16.2	26.5	30.6	34.9	37.7	36.3	33.8	29.0	34.5
Related business	8.5	1.7	7.2	6.1	4.9	3.5	5.3	4.7	4.4
Total	24.7	28.2	37.8	41.0	42.6	39.8	39.1	33.7	38.9
Family business:									
Medical and dental	3.1	2.1	3.6	1.4	1.2	2.1	1.9	3.4	2.1
Shopping	18.1	23.5	15.2	16.2	13.7	14.1	14.2	17.7	14.9
Other	20.3	18.9	18.5	14.1	14.3	15.0	15.1	12.0	15.1
Total	41.5	44.5	37.3	31.7	29.2	31.2	31.2	33.1	32.1
Educational, civic, and religious	11.6	8.7	11.7	10.4	11.2	12.9	13.1	11.1	12.3
Social and recreational:									
Vacations	[2]	[2]	[2]	[2]	[2]	0.1	[2]	[2]	[2]
Pleasure rides	3.2	2.1	1.6	2.8	2.9	1.9	1.6	3.2	2.1
Other	19.0	16.5	11.6	14.1	14.1	14.1	15.0	18.9	14.6
Total	22.2	18.6	13.2	16.9	17.0	16.1	16.6	22.1	16.7
All purposes	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Nationwide automobile-use study, spring 1961—persons living in places having a population of 100,000 or more.
[2] Less than 0.1.

Table 6.—Percentage distribution of automobile trips and travel to downtown shopping areas, by major purpose [1]

Distance from residence, one way	Purpose of trips and travel							
	Earning a living		Family business		Educational, civic, and religious		Social and recreational	
	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel
Miles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Less than 2	28.6	29.7	62.8	61.7	1.1	0.9	7.5	7.7
2.0–4.9	58.1	57.2	30.3	30.6	2.5	2.6	9.1	9.6
5.0–9.9	51.7	54.5	36.3	34.3	1.7	1.8	10.3	9.4
10.0 or more	50.9	52.5	35.5	34.3	2.2	1.8	11.4	11.4
Total	51.9	53.5	36.3	34.0	2.1	1.9	9.7	10.6

[1] Nationwide automobile-use study, spring 1961—persons living in places having a population of 100,000 or more.

Table 7.—Percentage of all automobile trips, by purpose, to downtown area [1]

Purpose of trip	Percent to downtown business area
Earning a living:	
To and from work	13.7
Related business	20.5
Total	14.5
Family business:	
Medical and dental	13.8
Shopping	13.1
Other	16.6
Total	14.5
Educational, civic, and religious	2.6
Social and recreational	6.7
All purposes	12.0

[1] National automobile-use study, spring 1961—persons living in places having population of 100,000 or more.

into any situation. Income is an important factor both in the choice of methods of going to work and in the distance between the worker's home and his place of employment. As shown in table 4 and figure 1, when family income was less than $1,000, more than half of all workers walked or bicycled to work. Possibly most of this group of families were domiciled at the job site. The data, however, do not show this. Almost 72 percent of the workers in the next income bracket ($1,000 to $1,999) used public transportation—a very substantial shift. The percentage of workers using public transportation dropped rather sharply until family income reached $10,000, at which point it increased somewhat.

As the size of family income increased, the choice of the automobile as a commuting mode increased and 63 percent of the families having incomes of $15,000 or more used automobiles for trips to and from work. Surprisingly, more than 20 percent of the families having the lowest incomes commuted by automobile, more than three-fourths of them driving. The more use of automobiles for home-to-work transportation as the family income exceeded $5,000 may be noted in figure 1, which shows the distribution of persons in each income group according to the mode of home-to work transportation. Except for the group of families having incomes of less than $1,000, in which 54 percent of the workers reported walking or bicycling to work, the groups were relatively consistent in the proportion of walkers. One might conclude that income above the $1,000 level is not closely related to walking to work.

Purpose of Trips

Family income is generally considered an important determinant in the use of automobiles. The distribution of automobile trips, by purpose, for the different income groups is shown in table 5. The proportion of trips related to earning a living rose from 24.7 percent for the family group having incomes of less than $1,000 to 42.6 percent for the group having incomes of $4,000 to $4,999 and then dropped off steadily to 33.7 percent for the group having incomes of $15,000 or more. Persons in the family groups having incomes of less than $2,000 made a larger proportion of the trips for purposes of family business than persons in other income groups. Social and recreational purposes accounted for 16.7 percent of all trips; groups having the highest and lowest income reported 22 percent of the trips were made for such purposes.

Urban planners are constantly aware of the problems related to keeping traffic flowing to and in central business districts. Although the tremendous buildup of suburban shopping and medical areas has reduced the attraction to a downtown shopping area, the downtown areas still attract people for a variety of reasons. Distribution of trips and travel by automobile to downtown shopping areas is shown in table 6, by purpose of trips. More than half of all trips and travel to the downtown shopping areas was made for purposes of earning a living; an additional one-third of the trips was made on family business. Only 2.1 percent of the trips and 1.9 percent of the travel were made for educational, civic, and religious purposes. Trips made for social and recreational purposes amounted to 9.7 percent of all trips and 10.6 percent of all travel. Persons who lived closer to downtown shopping areas—that is, less than 2 miles—made a smaller proportion of automobile trips to work and a larger proportion on family business than persons living farther from the downtown area.

The percentage of automobile trips to the downtown area, by purpose of the trip, was fairly substantial—12 percent—as shown in table 7. Of all trips related to earning a living, 14.5 percent were made to the downtown area. Trips on family business made to the downtown area also totaled 14.5 percent. Trips made for educational, civic, and religious purposes comprised less than 3 percent of those made to the downtown area; but 6.7 percent of the social and recreational trips were made to the downtown area.

NEW PUBLICATIONS

Highway Progress, 1965

The annual report of the Bureau of Public Roads, U.S. Department of Commerce, *Highway Progress, 1965*, contains a review of the accomplishments of Public Roads on the Federal-aid highway program and its many other activities during the fiscal year 1965. *Highway Progress, 1965*, is available from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for 40 cents a copy.

Included in the illustrated publication is a descriptive account of the progress made during fiscal year 1965 in construction of the National System of Interstate and Defense Highways and in improvement of primary highways, secondary roads, and urban arterials under the regular Federal-aid program. Also reported on at length are Public Roads efforts to meet the social responsibilities that face today's highway builder, and its activities and accomplishments in highway planning and design, urban transportation planning, highway beautification, safety, research and development, and management.

The report also describes the highway construction work undertaken directly by the Bureau of Public Roads in national forests and parks and on other Federal lands, and Public Roads activities in providing technical assistance to foreign countries to further the development of their highway organizations and roadbuilding programs. Included in the report's appendix are 19 statistical tables covering the progress and activities of the Federal-aid program during the fiscal year 1965. The data are reported for each State.

U.S. GOVERNMENT PRINTING OFFICE:1966

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–33 are available upon request addressed to Bureau of Public Roads, Washington, D.C., 20235.*

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington, D.C., 20402. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Annual Reports of the Bureau of Public Roads:
1960, 35 cents. 1963, 35 cents. 1964, 35 cents. 1965, 40 cents. (Other years are now out of print.)

REPORTS TO CONGRESS

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:
Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.
Supplementary Report, House Document No. 124 (1965). $1.00.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354 (1964). 45 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42 (1965). 20 cents.

PUBLICATIONS

A Quarter Century of Financing Municipal Highways, 1937–61, $1.00.

Accidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

Capacity Charts for the Hydraulic Design of Highway Culverts (Hydraulic Engineering Circular, No. 10) (1965). 65 cents.

Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

Design of Roadside Drainage Channels (1965). 40 cents.

Federal Laws, Regulations, and Other Material Relating to Highways (1960). $1.00.

Highway Bond Financing . . . An Analysis, 1950–1962. 35 cents.

Highway Finance 1921–1962 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

Highway Planning Map Manual (1963). $1.00.

Highway Planning Technical Reports—Creating, Organizing, and Reporting Highway Needs Studies (1964). 15 cents.

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds (1964). $1.00.

PUBLICATIONS—Continued

Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds (May 1965). 75 cents.

Highway Statistics (published annually since 1945):
1956, $1.00. 1957, $1.25. 1958, $1.00. 1959, $1.00. 1960, $1.25. 1962, $1.00. 1963, $1.00.

Highway Statistics, Summary to 1955. $1.00.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Highways and Economic and Social Changes (1964). $1.25.

Hydraulics of Bridge Waterways (1960). 40 cents.

Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). Out of print. Appendix, 70 cents.

Interstate System Route Log and Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal-Aid Construction, 2d ed. (1965). $1.75.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.
Part V—Traffic Controls for Highway Construction and Maintenance Operations (1963). 25 cents.

Opportunities for Young Engineers in the Bureau of Public Roads (1965). 30 cents.

Reinforced Concrete Pipe Culverts—Criteria for Structural Design and Installation (1963). 30 cents.

Road-User and Property Taxes on Selected Motor Vehicles, 1964. 45 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways (1958): a reference guide outline. 75 cents.

Standard Plans for Highway Bridges (1962):
Vol. I—Concrete Superstructures. $1.00.
Vol. II—Structural Steel Superstructures. $1.00.
Vol. III—Timber Bridges. $1.00.
Vol. IV—Typical Continuous Bridges. $1.00.
Vol. V—Typical Pedestrian Bridges. $1.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Economic Studies in Urban Transportation Planning (1965). 45 cents.

Traffic Assignment and Distribution for Small Urban Areas (1965). $1.00.

Traffic Assignment Manual (1964). $1.50.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Transition Curves for Highways (1940). $1.75.

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